IRON AND STEEL BRIDGES
AND VIADUCTS

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A PRACTICAL TREATISE UPON THEIR CONSTRUCTION

FOR THE USE OF

ENGINEERS, DRAUGHTSMEN, AND STUDENTS

BY

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PREFACE.

In this work my endeavour has been to show, first, the physical properties of different forms of iron and steel, and the mathematical and mechanical principles upon which the Construction of Iron and Steel Bridges is based; to deduce formulæ from the theoretical investigations of the relations of stress to resistance, suitable for every-day use in designing bridges; and to illustrate by example the application of these formulæ in practice.

So far as it may be done by description, I have tried to take my readers through the drawing office routine in a Bridge Engineer's establishment, and thus show the ultimate bearing of the theoretical reasoning upon the practical designing of the work.

The arrangement of such details as joints and connections has also received very careful consideration, for it is in determining these that the draughtsman is often confronted with awkward problems, especially in light work where there is not much room for joint attachments.

The construction of bridge flooring is dealt with very

fully, as also the provision to be made in bearings for expansion and contraction, and for deflection of girders.

The latter part of the volume is devoted to examples of different types of Iron and Steel Bridges, including all those forms in general use and of acknowledged utility.

In a concluding Chapter particulars are given of Brick, Concrete, and Masonry Piers and Abutments to carry iron and steel superstructures.

FRANCIS CAMPIN.

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IRON AND STEEL BRIDGES AND VIADUCTS.

INTRODUCTION.

ALTHOUGH a history of bridge-building, interesting and instructive as it is, would be out of place in these pages, it is desirable to glance lightly at the evolution of modern typical bridges from the cruder structures of earlier times.

There are two principal attributes which command admiration—one, excellence of design, and the other magnificence of extent; the occurrence of the latter will depend upon local requirements, and that of the former be controlled by commercial considerations and the ability of the designer.

At the dawn of bridge construction the materials available were few, and the forces to which they were subject very imperfectly understood. Strong vegetable fibres supplied the ropes for the rough suspension bridges of South America; long pieces of timber served for the support of straight bridges; and stone supplied the material for lintel bridges and those carried on arches, for the latter of which bricks were also available.

The properties of each material would naturally decide the class of work to which it was applicable. The tensile strength of the ropes and the facility with which they could be drawn across chasms at once suggest their value for suspension bridges; two or three ropes with a grooved block resting upon them, so that it can be dragged back and forth from side to side by lighter ropes, and having a basket suspended from it for the reception of passengers and produce, would meet the requirements of a primitive community in a country where the distances to be bridged are frequently considerable. For shorter openings the straight girder bridge, with a rigid platform built on it, would afford better accommodation for heavy traffic, and also allow the passage of cattle and wagons.

Some large bridges in which the carrying members are stone lintels exist in China, but wood is far more suitable for girder-shaped structures, because the resistance of the latter to transverse stress can be calculated, and the quantity of material suited to the duties incumbent upon it, but stone —being practically inelastic—does not come within the scope of formulæ for transverse strength, so that in fact stone lintel construction is but guesswork. We find by experience that a certain piece of stone laid across an opening will carry the load which passes over, or rests upon it, but we do not know what excess of material is used, nor does this instance enable us to determine the sizes of lintels for larger openings or for different intensities of load.

The resistance of stone to crushing can, however, be ascertained by experiment, and therefore for arches and other structures where the stresses are compressive only this material may be scientifically adjusted in quantity and form to the requirements of the work.

The great durability of stone and brick gives these

materials a marked advantage over wood for permanent works, but with the advent of iron for bridge construction a saving in cost was anticipated, as this material could be made lighter than the stone, and the expense of carriage and labour reduced. The first transition we notice is from stone to cast iron—a bridge in that material being built at Colebrook Dale in 1779, and another at Sunderland in 1796.

Masonry arches had always been designed with a view to stability as distinct from strength; the cross-sectional area always presents ample surface to provide against the intensity of the thrust, if the arch is thick enough to keep the line of thrust properly within its substance; if this is not done, the voussoirs of the arch will turn over on their edges, unless the cementing material is adhesive enough to resist the disturbance. Following this principle, the cast-iron arch was first made with voussoirs of such outside dimensions as might have been used with stone, but they were really skeleton voussoirs, and thereby lightness was secured, while sufficient sectional area was retained to resist the thrust.

An improvement on this form of construction appeared in 1819, in Southwark Bridge, in which much flatter arches of I section were used, and this form has been generally adopted since for both iron and steel bridges.

In 1825 the Menai Suspension Bridge was completed, consisting of a roadway carried by vertical rods hung on to chains formed of long links of round iron bars, with eyes at each end, the iron in this case being wrought, not cast; and a similar construction was adopted for the Brighton Chain Pier, recently pulled down. The round iron links were in later structures superseded by flat bars rolled with wide ends to form the eyes, and the next step was the introduc-

tion of steel wire ropes, which have given very satisfactory results. These have been made twisted as in ordinary hemp ropes, and also with the wires laid parallel and kept together by rings at frequent intervals, whereby the stress on the wires is reduced if the connection is such that they are all equally loaded. We cannot, however, be sure of this condition, whereas it must obtain with the twisted rope, which is one point in its favour.

Wooden girders, beams, and trusses also gave place to cast-iron girders, and many elaborate experiments were conducted in order to ascertain practically the strength, deflection, and mode of rupture of such girder, and also the best proportions of cross-section to adopt, and cast iron also came into use for columns and stanchions.

Cast iron is especially adapted to resist compressive stress, its ultimate resistance to crushing being from 36 to 42 tons per sectional square inch, while its tensile strength is only from 5 to 7 tons for ordinary cast iron; it has reached 10 tons or more in special brands for the manufacture of ordnance, but that does not apply to girder work.

The tensile weakness of cast iron naturally turned the attention of engineers towards wrought iron, but at that time cost of production was high; but yet for large spans it was obviously imprudent to trust to cast iron, and principally because cast iron breaks with very little warning; it will not bend, and wrought iron may be crippled or bent without actually giving way. Another objection to long cast-iron girders was that, as they will not bear the operation of rivetting, they must be bolted together with bolts truly turned to fit drilled holes, and this involves much expensive labour. When the Britannia Tubular Bridge was projected—to carry a railway across the Menai Straits—

cast iron was first thought of as the material, the original crude idea being to place two gigantic cast-iron flanged girders side by side to form a tube, through which the trains would pass, over a permanent way laid on the bottom flanges. This bridge comprises four spans, of which two are 460 feet each, and it was very properly decided that wrought iron should be the material used in its construction; so the bridge assumed the form of two rectangular tubes side by side, built up of wrought-iron plates rivetted together and stiffened by tee and angle irons rivetted on to the side webs; the roof and floor constitute the flanges, and are cellular in form. This work may be regarded as that which proved the applicability and value of wrought iron for large bridges; and although, compared with modern structures, it is excessively heavy, yet at the time of its erection (1850) it was a masterpiece. This proving a success, another bridge on the same principle (but with cellular top flange only) was constructed over the St. Lawrence at Montreal nine years later.

The tubular form, however, was found to be unnecessarily heavy, and the form generally adopted became that comprising two or more longitudinal girders—girders supporting a roadway between them, or carried upon their top flanges. The main girders of the bridges were and are made with plate webs up to moderate spans; but for long spans—trussed, lattice, or triangular—open webs have come into general use. With the general adoption of wrought iron for straight girder bridges came also its application to arch bridges, although its compressive resistance is not equal to that of cast iron.

Certainly, wrought iron is better qualified than cast to resist the violent vibrations and sudden jerks to which railway bridges are subject, and this may be a sufficient

reason for its adoption even for arches, when the latter are used for such structures; but it certainly seems to me that, with the increasing use of wrought iron, cast iron has become unduly discredited.

Although the tubular section of bridge was not adopted in general practice, yet for some years the box or tubular form of the top flange was in evidence, the idea being to stiffen it against lateral or vertical crippling, but except for long spans this has quite given place to plain plate flanges; in fact, the tendency has been towards simplifying construction in all directions.

The introduction of rolled steel as a material for bridge construction in place of wrought iron does not seem so wide a step as that from cast to wrought iron; but when this was first attempted there was much doubt as to the reliability of steel then put on the market. Some samples of mild steel would crack without any apparent reason, and while such a thing could occur no confidence could be placed in the material.

The structural, or molecular, difference between iron and steel is this—the former is an element, consists, that is, of one kind of matter only, and the latter is a compound of iron and carbon; the former is fibrous, and the latter without grain.

The cracking referred to must be caused by a want of homogeneity in the metal, and this may easily occur in some processes of manufacture, but this defect being obviated, the material can be employed with as much confidence as wrought iron.

There are several methods of making steel, which in the widest sense may be described as an alloy of iron with carbon, manganese, or some other substance which increases its strength and gives it the property of hardening when

heated to a high temperature and suddenly quenched in water or other cooling medium. The method formerly in general use is known as "cementation," and consists in placing small bars of wrought iron in charcoal in boxes, and keeping these exposed for long periods to a temperature of about 2,142° Fahr.—which approaches a white heat—in furnaces of special construction. The bars for conversion are either hammered or rolled bars; the former are preferred; and in size they are about 3 inches in width, $\frac{5}{8}$ to $\frac{3}{4}$ inch in thickness, and 6 to 12 feet long. The boxes or pots are charged by first covering the bottom with little lumps of charcoal to a depth of about $\frac{1}{2}$ inch, upon which are placed bars of iron flat side down, with spaces between for charcoal. A second layer of charcoal is spread over these bars, and on that comes another layer of bars, and so on until the required charge is introduced, the top layer being of charcoal. The whole is covered in with a plaster made of mud which collects in the troughs of grindstones, which consists of siliceous matters and particles of partially oxidized steel, a mixture which—under the effects of the heat of the converting furnaces—fuses, and forms a pasty mass or glazed air-tight covering to the pots.

The furnace when charged is fired and the temperature gradually raised, till after 24 hours a red-heat is reached, and in 48 hours the glowing heat necessary for conversion is reached. This is maintained until the desired degree of conversion is reached, as determined by the examination of trial bars withdrawn from time to time. The process may last from seven to nine days.

When cold the charge is withdrawn and the bars broken up and sorted, according to the appearance of the fractures; the resulting steel contains from 0·5 per cent. to 1·5 per cent. carbon, according to the duration of the treatment.

The bars are piled together, heated and hammered, but after hammering and rolling are by no means homogeneous, and the impurities mechanically incorporated can only be removed by fusion, which is effected in crucibles by a process which was introduced into Sheffield in 1740, and has remained in use practically unaltered since then. The ordinary crucibles are from 16 to 18 inches high, and 6 to 7 inches diameter at the mouth; they are made of mixtures of fire-clays with potsherds, coke dust and graphite, and are covered by lids during the fusion of the metal; when the fusion is complete the steel is poured into "ingot-moulds," and has now become "cast steel." The ingots are subsequently worked under the hammer and in the rolls to the required forms.

This method of producing steel involves a great deal of labour and the consumption of large quantities of fuel, thus bringing the price too high for a bridge material, and further such high percentages of carbon are not required in the "mild steel" used for girders; the process, however, very clearly shows the change that is required to convert wrought iron to steel.

In another process crucible steel is produced by the direct fusion of bar iron with charcoal and spiegeleisen, which last is a highly manganiferous pig-iron. For the low grade steels required for constructional purposes no charcoal is necessary, sufficient carburization being obtained by the small addition of spiegeleisen made toward the end of the melting, with the small proportion of carbon absorbed from the plumbago pots in which the metal is melted; these pots are made with only enough clay to give the necessary strength to bear the pressure of the molten metal and the manipulation to which they are subjected.

Steel may be also obtained by fusing pig-iron with rich

ores, or oxides of iron, and the most important is the open-hearth direct process. The Uchatius and Ellerhausen processes have been discontinued in England on account chiefly of the inequality of the material produced, and the Siemens-Martin method is now in general use.

In this last process wrought iron or scrap steel is melted in a bath of molten pig-iron on the hearth of a regenerative gas furnace. The furnace, being already heated from the working off of a previous charge, has the bottom levelled and repaired, and then the charge is introduced, first putting on the hearth from 15 to 20 per cent. of pig-iron, to which is added about 66 per cent. of clean scrap steel and about 15 per cent. of old rails. These proportions will vary with the nature of the materials obtainable at the time and according to the conditions of the furnace.

After complete fusion has occurred samples of the metal are withdrawn from the furnace in a small ladle, and hammered and broken to test their malleability and roughness, and to examine the fracture and make a rapid chemical analysis to determine the amount of carbon present in the metal. When the tests indicate that a sufficiently low temper has been reached by the metal, then about 1 per cent. of spiegel, containing 8 to 15 per cent. of manganese, and about 3 per cent. of a ferro-manganese containing from 60 to 80 per cent. of manganese, is added through the side doors of the furnace. A few minutes serve to melt this, during which time the metal is thoroughly stirred to mix it, and it is then tapped as quickly as possible to prevent serious loss of the manganese in the slag.

In the SIEMENS open-hearth process pig-iron, scrap steel, and the purer hæmatite iron ores are fused together. The pig-iron is first decarburized by the hæmatite ore, and subsequently recarburized to the required degree by adding

spiegel or ferro-manganese, as in the method previously described.

The BESSEMER process is peculiarly simple and elegant. Pig-iron in a melted condition is run into a converter, which is a nearly cylindrical vessel with a short open neck at the top; into the bottom of the converter is forced a blast of air, which passes through the charge, decarburizing and refining it by oxidation and combustion of its carbon, silicon, and manganese, with development of sufficient heat to keep the metal in a fluid condition to the end of the process; the metal so decarburized is then recarburized to the required degree by the addition of the proper percentage of spiegeleisen or ferro-manganese.

The converter is usually mounted on trunnions, so that it can be partially rotated by suitable gearing, and laid in position to receive its liquid charge, then righted for the blowing process, which lasts about eighteen minutes. After this it is again laid back to receive the spiegel, and subsequently turned further over to pour its charge into a ladle in which it is carried to the ingot-moulds. This material is sometimes called "ingot-iron." In the ordinary Bessemer process the converter is lined with a siliceous sandstone called "ganister," and so the whole of the sulphur and phosphorus existing in the pig-iron remains in the steel produced; therefore the purer pig-irons only should be used with this lining: this is known as the "acid process." By a modification introduced by Messrs. Thomas and Gilchrist the converter is lined with dolomitic lining, which is highly basic, and therefore eliminates a large proportion of the phosphorus and sulphur, and thus allows for the use of phosphoric pig-irons; this is known as the "basic process."

The Bessemer process shows a remarkable contrast to puddling processes in point of time consumed in the produc-

tion of steel from pig-iron. As much as 2,830 tons of steel has been stated to have been produced from a single pair of converters, each dealing with a charge of 8 tons at a time.

In the basic process some well-burnt lime—about 15 to 20 per cent. of the weight of the charge—is first introduced into the converter with a little coke breeze, and brought up to a bright glow by gently blowing through the converter, after which the charge of molten pig-iron is introduced, and the blow proceeded with at the higher pressure. The pressure of the blast is about 25 lbs. at the commencement, but is sometimes lowered as the process draws to a close: this, however, will of course vary with the weight of the charge over the tuyere openings in the bottom of the converter. The progress of the operation may be observed by watching the flames through a spectroscope, but with English irons this is not often found necessary to the practised eye.

The tensile strength, elasticity, and rigidity of steel vary with the temper as affected principally by the proportion of carbon present, but also in some degree by silicon, manganese, tungsten, phosphorus, and other matters: hammering, rolling, wire-drawing, and other mechanical treatment also increase the tensile resistance and ductility of the metal.

"Mild steel" containing from 0·05 to 0·02 per cent. of carbon will weld, but will not temper, and breaks under tensile stress varying from 23 to 32 tons per sectional square inch, with an elongation before fracture of from 25 to 30 per cent. in a length of 8 inches.

Steel containing from 0·20 to 0·35 per cent. carbon has a tensile strength from 30 to 38 tons per square inch.

The elastic limit in ratio to the ultimate strength of steel

is high. Metal having a tensile resistance of 24 tons will not stretch permanently under a stress less than 13 or 14 tons. With tensile strength of 28 tons the elastic limit will not be under 15 tons, and in hard-drawn wire an elastic limit of from 45 to 50 tons is obtainable.

The advantages in using mild steel over wrought iron are a great saving in weight of permanent structure, and fewer joints, as steel plates and bars can be rolled in much longer lengths than iron, thus saving joint plates.

Looking forward, aluminium seems to be the ideal metal for bridge structures, possessing as it does great tensile strength with lightness, and, in a very high degree, resistance to corrosion, but its adoption for this purpose awaits further commercial developments in connection with production.

CHAPTER I.

THE THEORY OF STRESS AND RESISTANCE IN GIRDER BRIDGES.

THE constitution of the universe being such that some force is always in action, it follows as an axiom that if any particle or mass of matter is at rest, there must be at least two forces acting upon it, and they must be equal in intensity and opposite in direction; for were one greater than the other motion would commence, the weaker force being overcome, and if they are not opposite, their combined action will cause lateral movement. If a number of forces act upon a body, and that body is at rest, all the forces acting on one side of it must be counterbalanced by those acting on the other side. If any of the forces which are holding a body in equilibrium are varied, displacement of that body will ensue, and a new position of equilibrium be established.

A simple experiment may be made with a letter-weighing machine, which consists of a scale-pan attached to a vertical stem, sliding in a stand and supported by a spiral spring. When there is nothing on the scale pan and it is at rest, the two forces in action are gravitative attraction on the pan and upward pressure of the spring; let the pan be lifted off the spring, then the latter will be free from external weight; if laid upon its side each coil will press on

the supporting surface, and be flattened out of circular form by its own weight, and if it is restored to its vertical position it will be shortened in height by its own weight before it comes to rest. If the scale pan be now placed upon it, it will be compressed by the weight of the pan, until it presses upwards with an elastic force equal to that weight, when motion will cease, because the forces in action are equal and opposite, but some movement has occurred before this condition is reached.

If now any weight within the strength of the spring—say one ounce—is placed upon it, the spring will be further compressed through some definite distance before rest again occurs. If twice this weight is placed upon the scale the compression will be twice as much, and in short the compression will, so long as the elasticity of the material remains unimpaired, be proportional to the force producing it. If the pan were hung to a suspended spring, then on placing weights in it the spring would be extended, and the extension would be proportional to the weight or force producing it. This is a most important physical law, and is known as Dr. Hooke's law from the first enunciator, who gives it the form of "as is the extension so is the force." I have here taken a spiral spring for experiment because it has a long range of compression and extension which is readily measured, but the same law applies to the direct extension and compression of bodies.

All material bodies are built up of molecules, or groups of atoms, which may be similar in character or different, but have one common condition, which is that the atoms are not in contact with each other, but hold positions dependent upon the internal and external forces to which any particular body may be subject. The molecules thus situated are balanced, as it were, between their reciprocal

attraction of cohesion drawing them together, and some force tending to separate them: this latter may be heat. Now, an external force may act to compress, or bring the molecules closer together, or to drag them farther apart, than they were in their normal condition. So long as the alteration of the molecular distances does not exceed the range of the normal forces acting upon them in an unstrained condition, Hooke's law will apply, but when this range is exceeded permanent displacement of the molecules occurs, and the shape or bulk of the body is altered. It is then said that the "limit of elasticity" has been exceeded, and the material is injured if not absolutely broken.

Upon the basis of these simple facts rests the whole theory of stress and resistance, and a little mathematical knowledge enables us to ascertain the relations existing between them, and I wish to impress upon the student that mathematical investigation in these matters is continuous; it is merely the application of ordinary reasoning, and does not consist of abrupt and disconnected formulæ and methods, labelled with the names of those who have first brought them into technical handbooks. This I point out because this pigeon-holing of rules is confusing, inasmuch as it leads the student to think that each case must have its own exclusive theory, instead of recognising the fact that logical reasoning flows equably over all the cases and all variations which now come under his notice, or which can at any future time call for his consideration.

The stresses to which iron and steel bridges are subject are five—tension, compression, transverse stress, shearing stress, and torsion; which may be analysed into the simple stress, tension.

TENSION is the only stress which can be regarded as

simple, that is, causing only one kind of disturbance in the element to which it is applied. The length of a bar does not affect its tensile resistance, which varies simply as its cross-sectional area; thus, if $a =$ the cross-sectional area of a bar, of which the ultimate tensile resistance is 30 tons per square inch, the breaking stress of that bar will be $= 30$ tons $\times\ a$ square inches. Failure under tensile stress is due to the molecules of the bar being drawn so far apart as to fall out of the range of their mutual cohesive attractions.

Compression is apparently a simple stress, but a little consideration will show that failure under a force producing it can only occur by the setting up of other forms of stress, for the atoms of matter being accepted as indestructible, the greatest pressure brought upon a group of them can only bring them into actual contact and so form an unyielding mass. Fracture under compression, in short blocks, must then be due to some lateral displacement of the molecules. In Figs. 1 to 4 different modes of failure are shown. While considering these illustrations we must keep in mind the structure of the block *a b c d*, which is under pressure from a force, *P*, and this structure comprises a mass of molecules held together by cohesive force interacting amongst them in all directions.

The materials of commerce are never homogeneous, and so there will be some planes of section in which the cohesive forces are weaker than in others, and in these planes of least resistance rupture will naturally occur. The form of the block is maintained by the resistance of the vertical forces to descent of the upper parts, and by the lateral cohesion which prevents the sides from falling out. If a pressure coming on the block (Fig. 1) tends to push a piece off along the plane *e f*, this movement will be resisted by

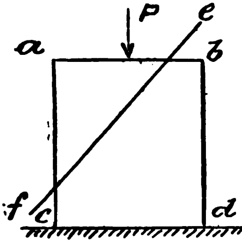


Fig. 1.

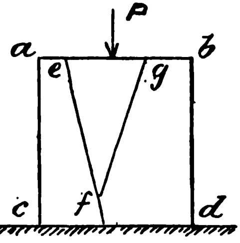


Fig. 2.

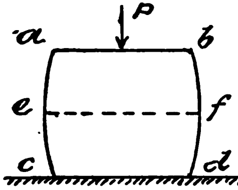


Fig. 3.

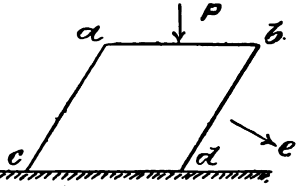


Fig. 4.

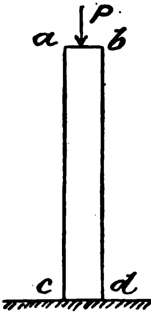


Fig. 5.

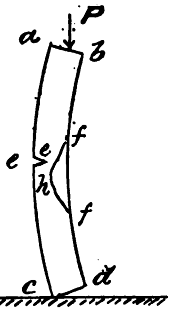


Fig. 6.

an upward pull of cohesive force, but ultimately failure will occur by shearing along the plane *e f*, if that is the plane of least resistance. In Fig. 2 the condition assumed is, that there are two equally weak planes of resistance, *e f* and *g f*, the matter both inside and outside of these planes having a superior cohesion to that obtaining along these planes; then the central part *e f g*, acted upon by the force *P*, will act as a wedge to rend the block asunder, and it will probably break into three principal pieces, and several fragments from the edges.

In Fig. 3 a different result is shown; the block *a b c d* is assumed to be originally cylindrical, and so nearly uniform in constitution as to exhibit no plane of distinguishable weakness: then the pressure *P*, acting equally upon its top surface, will cause it to bulge, and this being continued until the lateral cohesion in the plane (*e f*) of greatest expansion is overcome, it will settle down and split round the edges.

Another form of settlement is shown in Fig. 4, which represents a cubical block, *a b c d*, which has commenced to yield by the comparative weakness of the parts near, and parallel to the side *b d*, and thus led to the progressive sliding, horizontally, of the layers of molecules until the block fell over in the direction shown by the arrow *e*.

It is evident that when there are so many different ways of failure, and we cannot from theorising determine which will occur in any particular case, experiment alone can help us, and from these certain data are derived, from which minimum strengths are ascertainable, and in designing bridges it is upon minimum strengths that we must base our calculations.

We now come to another case, or rather series of cases, in which the elements to be considered are long enough in proportion to their diameters to bend before breaking. If

bending occurs there must be transverse stress, but this stress occurring in a column is not exactly the same in its effect as that brought upon a girder supported at each end, and carrying a load distributed upon it horizontally. If the column were bedded upon a true surface, and the load upon it evenly distributed, and the column itself absolutely homogeneous, it would fail by bulging and splitting radially, but as this condition does not obtain, and one part of the column offers less resistance than the rest, the column will bend, and become concave on the side to which the weak spot is nearest. Now, it may happen that, although the column is bent, the line of pressure may not pass outside the horizontal section, and in that case the whole section will be in compression, but the intensity of the compression will vary, being a maximum on the concave side of the column.

If, however, the curvature is such that the line of pressure passes outside the horizontal section, then the convex side of the column will be in tension, and the stresses will more nearly approach those on a girder carrying a transverse load, but cannot be analogous to them on account of the different mode of support.

In Fig. 5, *a*, *b*, *c*, *d* represents a column sufficiently long in proportion to its diameter to bend before breaking, and Fig. 6 shows the same column on the point of breaking under a vertical load, *P*. The mode of fracture will be by tearing asunder at *e e*, and splitting out one or more wedge-shaped pieces at *f h f*. The rupture at *e e* is in tension, that at *f h f* shearing.

TRANSVERSE STRESS.—*A B*, Fig. 7, represents a horizontal beam supported on bearings at *C* and *D*, and, unstrained, as its weight is neglected in this illustration.

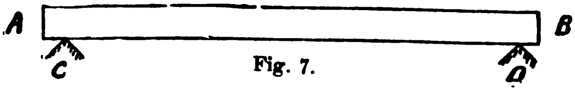


Fig. 7.

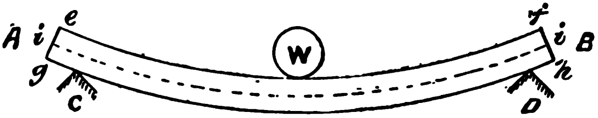


Fig. 8.

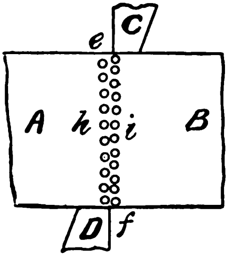


Fig. 9.

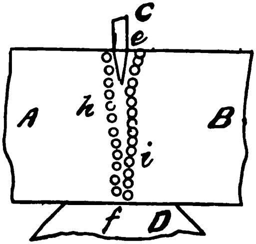


Fig. 10.

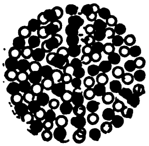


Fig. 11.

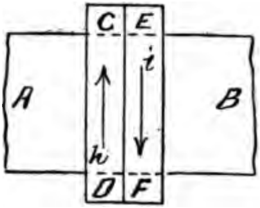


Fig. 12.

Fig. 8 shows the same beam loaded with, and deflected by, a weight, *W*. The ends of the beam, originally parallel, are now inclined to each other; the upper surface, *e f*, is shortened, and therefore compressed, and the lower surface, *g h*, is extended; the top surface being in compression, and the bottom in tension, there must be some horizontal plane in the beam where there is no tension. This is indicated by the dotted line, *i i;* from this plane the stresses per sectional square inch will increase, in compression upwards, and in tension downwards, to the surfaces of the beam. The varying values and their summation will be dealt with further on; so far, transverse stress is shown as resolved into tension and compression.

SHEARING STRESS tends to destroy the continuity of a body by sliding one part off the rest, as shown in Fig. 9, in which *A B* is part of a bar submitted to the action of the shearing edges *C* and *D*; these edges are nearly rectangular, and while the part *A* of the bar is supported upon the edge *D*, the upper shear *C* forces the part *B* down, and severs the bar through the plane *e f*. The small circles *h* and *i* on each side of this plane represent molecules, and it will be seen that in the sliding of one layer over another it is the cohesive, or tensile, attraction that is overcome.

Shearing force must not be confused with cutting action, which is shown in Fig. 10; here the bar *A B* rests upon a block *D*, and is acted upon by a cutting edge *C*, which divides it through the plane *e f*. This cutting edge is so fine that it enters between two layers of molecules and wedges them apart, overcoming their tensile cohesion by forcing the molecules on either side beyond the sphere of action.

TORSION is generally treated as a form of shearing stress, and under certain conditions this view will be correct. In Fig. 11 is shown a cross section of a cylindrical bar, consisting of molecules, represented by small circles. The circles shown in lines represent the position of the molecules before torsional stress is applied; the circles filled in show the displacement occurring when torsion is applied, as indicated in Fig. 12. In this *A B* is a side elevation of the shaft, and upon it are securely fastened two rings, *C D* and *E F*, close together. Now, if these two rings are forcibly turned in opposite directions, as shown by the arrows *h* and *i*, shearing stress comes into action, and the molecules are pulled away from each other, so this force is resolved into tension. When the opposing forces are at a distance, the lines of molecules are moved into spiral lines, from their normal positions parallel to the axis of the bar, and thereby extended, and therefore placed under tensile stress.

Having described the character of the various stresses to which materials used in bridge construction are subject, the next step is to show how the intensities of such stresses are determined, and also the resistances.

There are two ordinary methods by which stresses are determined—the "parallelogram of forces," and the "principle of moments," and one of these can be used to check the other. These are both so exceedingly simple that their comprehension calls for no knowledge of mathematics beyond arithmetic, and yet they are sufficient to deal with every case that comes under our notice in connection with the subject of this work.

The parallelogram of forces deals with the resolution of two forces, meeting at a given point, into one opposing force which shall hold them in equilibrium; the data re-

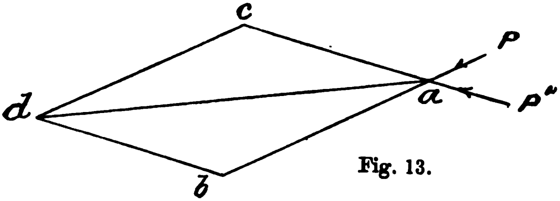


Fig. 13.

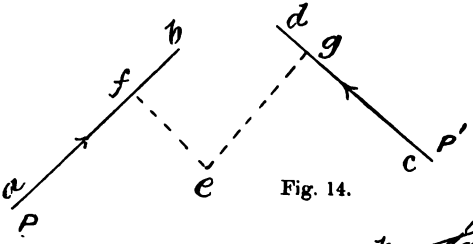


Fig. 14.

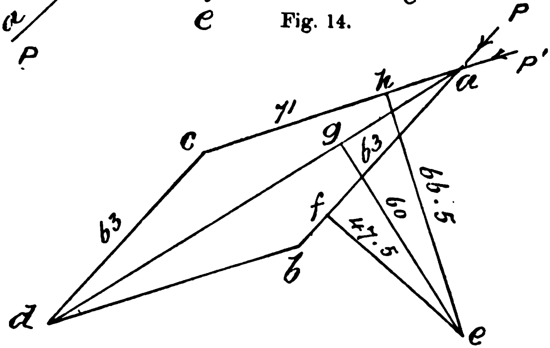
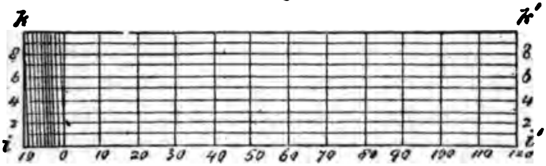


Fig. 15.



quired are the intensities and directions of the two forces, and the direction of the counterbalancing force.

The principle of moments deals with forces acting about a centre and tending to cause revolution about such centre; the moment of any force is equal to its intensity multiplied by the least distance of its direction from the point or centre about which it is acting. If a number of forces are acting about one point, summations of those acting in different directions will solve the question of equilibrium of the whole, and if several forces are acting at a point, the resultant force can be determined by a series of parallelograms of force.

Let P and P' (Fig. 13) be two forces acting at a point a and in the directions shown by the arrows; produce the direction of the force P to b, making the length $a\,b$ represent, to some convenient scale, the intensity of the force P, and produce the direction of the force P' to c, making $a\,c$ equal to its intensity. Complete the parallelogram $a\,b\,d\,c$, and join $a\,d$; then the diagonal $a\,d$ will represent in direction and intensity the resultant of the forces P and P'. If it is assumed that the forces are expended in motion, we may conclude that the force P would cause motion from a to b, and P' would extend the distance traversed from b to d—equal to a to c—or both forces acting simultaneously would cause motion along the line $a\,d$ to the same ultimate point d.

Let there be a point e (Fig. 14) about which two forces, P and P', act in the directions shown by the arrows, neither direction passing through the point e. Draw the lines $a\,b$ and $c\,d$ as far as may be necessary in the directions of the two forces, and from the point e draw perpendiculars $e\,f$ and $e\,g$ to these lines, then the lengths of the perpendiculars will be the shortest distances of the directions of the forces

from the point about which they act. The moment of the force P will be $= P \times ef$, and that of $P' = P' \times eg$, and if these moments are equal the forces will be in equilibrium.

The method of proving the parallelogram of forces by the principle of moments is shown by the diagram, Fig. 15. The forces P, P', are 63 tons and 71 tons respectively, and are represented by the lines $a\,b$ and $a\,c$ to the scale of tons k-i'. Completing the parallelogram and measuring the resultant $a\,d$ we find its value to be 128·57 tons; if this resultant is the true equivalent to the forces P, P', then the moments of these two forces about any given point should be equal to the moment of the resultant force about the same point. Take any convenient point e as a centre of moments, and from it draw three perpendiculars, ef, eg, and eh, to the directions of the forces under consideration. Using the scale as a scale of feet, the lengths of these perpendiculars are found to be $ef = 47{\cdot}5$; $eg = 60$; and $eh = 66{\cdot}5$ feet. The moments of the forces P and P' are :—

Moment of force	$P = 63 \times 47{\cdot}5 =$	2992·5	foot	tons
,, ,,	$P' = 71 \times 66{\cdot}5 =$	4721·5	,,	,,
	Total . .	7714·0	,,	,,

The moment of forces of the resultant about the point e will be

$$= 128{\cdot}57 \times 60 = 7714{\cdot}2 \text{ foot tons,}$$

which is a very close approximation to the previous figures; as near as can be expected when the data are obtained by scaling the lengths.

There is this apparent disadvantage about working by scale—that such absolutely accurate results are not obtainable as when given numbers form the basis of the calculation, but in actual practice the discrepancies are so small

compared to the gross amounts as to be negligable without incurring any appreciable error.

Every precaution is of course to be taken to secure the closest obtainable approximation to correctness, and for this purpose the diagonal form of scale, as exemplified by $i\,k$, $i'\,k'$, is the clearest and most convenient. I will therefore describe its construction.

The line $i\,i'$ is divided up into equal parts, each representing ten tons each, and from these divisions lines are ruled at right angles to $i\,i'$ up to the boundary line $k\,k'$ of the scale. The first division point m is made the zero, and the scale numbered from it right and left.

The boundary line $i\,k$ is divided into ten equal parts, and through the divisions so found lines are drawn the whole length of the scale parallel to the lines $i\,i'$ and $k\,k'$. Towards the left from the point m the ten tons space is divided into ten parts, so that each will represent one ton. From the first of these smaller divisions, a diagonal line is drawn to the commencement of the scale at k, and from each of the other small divisions lines are drawn parallel to it, and terminating at the line $k\,k'$.

If, now, we want to take off, say, 53 tons in the dividers, we should place one point of the instrument on the 50 division to the right of m, and the other on the third division to the left. If, however, 53·1 tons were required, the dividers would be shifted to the first parallel line above $i\,i'$ and stretched from the 50 division on that line to the crossing of the diagonal line ruled up from the third division to the left of m; for 53·2 tons the second line up will be taken. As each diagonal line traverses one ton space in the whole depth of the scale, it is obvious that its distance from the line $i'\,k'$ increases by one tenth of a ton for every line above the lowest, and the same division will apply to its use as a

scale of feet, but if inches were required the line $i\,k$ would be divided into twelve equal parts. By making the scale sufficiently deep to divide into one hundred parts the insections of one diagonal line would increase at each intersecting line by one hundredth of a ton. A scale of twenty tons to the inch is large enough for all ordinary diagrams of stress.

It is a mere waste of time to carry out calculations beyond the limit of utility; thus, in settling sectional areas in stress, we should not in any case work to fractions of a ton, and therefore if such occur they can be dropped, and the unit figure increased by one.

In order that any beam or girder may support a load placed upon it, it is necessary that the moment of resistance of the beam shall be equal to the moment of stress derived from the weight of the load. The methods of determining these moments in the simplest manner will now be shown.

There are certain letters of the alphabet which will be used throughout the investigations which follow, with the same significance in every case. They are : $W =$ total load on a girder ; $\mathrm{W} =$ local load on ditto ; $w =$ load per lineal unit of span distributed over a girder ; $l =$ span of girder ; $d =$ depth ; $b =$ breadth ; $M =$ moment either of stress or resistance ; $S =$ total stress on a transverse sectional area of a structural element ; $s =$ stress per square unit of the same ; $T =$ tension ; $C =$ compression ; $t =$ thickness. In indicating the character of stress $+$ will be used for compression and $-$ for tension ; the reaction of a pier or abutment supporting a girder $= R$; and $x =$ the horizontal distance from a given point to another point at which stress is required to be determined.

The various terms in each equation or formula must be taken in the same name ; thus, if l is given in feet, x and d

must also be in feet; and if W, w, or w are in tons, S, T, and C will also be in tons, and M in foot tons; if the first measurement is in inches, the others must also be in inches, and if the loads are in pounds, the stresses will work out in pounds.

In Fig. 16, $A\,B\,C\,D$ is a side view of a straight beam of rectangular cross section, assumed to be supported along its whole length, so that no transverse stress can come upon it. Let two vertical, parallel lines, $e\,e'$ and $f\,f'$, be drawn to represent the edges of two imaginary planes intersecting the beam at right angles to its length; now let the support between A and B be withdrawn, so that the beam is left supported only at the points A and B; it will be deflected by its own weight, and assume a form shown to an exaggerated scale in Fig. 17. The bottom layer $A\,B$ of the beam becomes extended, and the top layer $C\,D$ compressed; the line $e\,e'$ is no longer parallel to $f\,f'$. Midway between the compressed and extended boundary layers, $A\,B$ and $C\,D$, lies a layer $i\,i$, or rather a surface of division between the contrary stresses, which is termed the "neutral surface" of the beam; it cuts the planes $e\,e'$ and $f\,f'$ in h and h', which represent the ends of lines crossing those planes, and which are the "neutral axes" of those planes. Through the point h draw the straight line $f^2\,f^3$, parallel to $f\,f'$. The neutral surface $i\,i$, having no stress upon it, preserves the same length as the beam had when it carried no weight; $f^2\,e$ shows the distance through which the top layer has been compressed in the length of $h\,h'$, and $e'\,f^3$ shows the extension of the bottom layer in the same distance.

If the condition of any intermediate layer $k\,k$, or $m\,m$, is examined, it is found that its compression, or extension, as the case may be, is less than that of the layers outside eater than that of those between it and the neutral

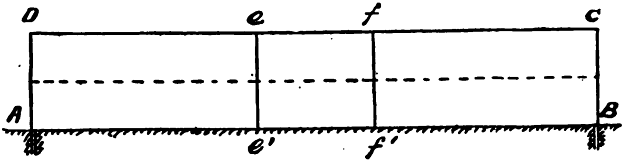


Fig. 16.

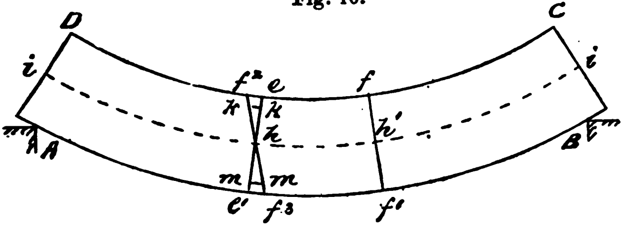


Fig. 17.

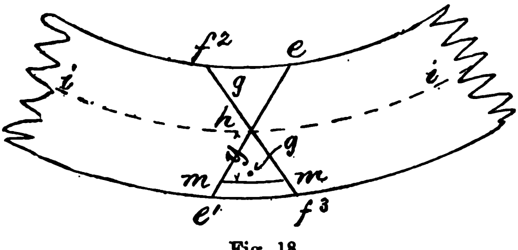


Fig. 18.

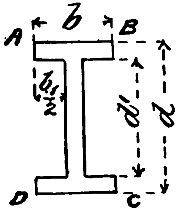


Fig. 19.

surface; in fact, the extensions and compressions vary from nothing at the neutral surface to maxima at the bottom and top of the beam. As Dr. Hooke's law shows that the resistance varies as the extension, these different layers will exert different resistances, producing moments of resistance about the neutral axis. In the larger diagram, Fig. 18, using the same letters, $e' f^3$ and $f^2 e$ represent the maximum of extension and compression, and in calculating the strength of a beam we must obviously first decide what maximum stress we are going to allow, say, per square inch of transverse sectional area, and this is to be taken as corresponding to the value of $e' f^3$. Taking one-fifth the breaking weight as safe working stress, then iron breaking at 22 tons gives the working value of $s = 22 \div 5 = 4{\cdot}4$ tons per sectional square inch; for steel breaking at 30 tons, the value of $s = 30 \div 5 = 6$ tons per sectional square inch.

As $d =$ the depth of the beam, the distance of the outside layers from the neutral axis is $\frac{d}{2}$, and as the extensions vary as the distances from that axis, the resistance s' of any layer $m\,m$, distant y from the neutral axis, will be

$$s' = s \times \frac{y}{\frac{d}{2}} = \frac{2\,s\,.\,y\,.}{d}$$

in tons per sectional square inch. If $b =$ breadth of the beam and $t =$ the thickness of a layer $m\,m$, then the total resistance of that layer will be,

$$S = \frac{2\,.\,s\,.\,y}{d} \times b \times t\,. -$$

and its moment of resistance about the neutral axis

$$M = S \times y = \frac{2\,.\,s\,.\,b\,.\,t}{d} \times y^2$$

If, however, the layer *m m* has any thickness, its outer surface will exert a higher resistance than its inner surface; the total moment cannot, therefore, be reached by adding together those of the component layers, and some means must be found to determine the sum of the resistances without assigning any definite thickness to the layers.

The stresses above and below the neutral surface are symmetrical, because the extensions and compressions follow the same ratio to a given length, so long as the limit of elasticity is not exceeded; therefore it is only necessary to deal with one half of the section for the present purpose, the total moment of resistance being found by doubling that of the half section chosen. The resistances of all the layers which constitute the lower half of the beam will be represented by lines drawn parallel to $e' f^3$ in the triangle $e' h f^3$, and as these lines must touch each other, the resistance of the half section of the beam is equal to the area of the triangle $e' h f^3$ multiplied by the breadth of the beam. The area of a triangle is equal to its base multiplied by half its height; the height in this case is $\frac{d}{2}$, and therefore the area of the triangle will be—

$$= s \times \frac{d}{4}$$

The moment of this force is found by multiplying it by the distance of its centre of gravity from the point *h*, about which it acts; the centre of gravity of an isosceles triangle is two-thirds of its height down from the apex, therefore the distance at which the summed forces act about the central axis is—

$$= \frac{d}{2} \times \frac{2}{3} = \frac{d}{3}$$

and multiplying by the breadth of the beam, the moment of resistance of one half is found to be—

$$= \frac{s \cdot d}{4} \times \frac{d}{3} \times b = \frac{s \cdot d^2 \cdot b}{12}$$

this multiplied by two gives the "MOMENT OF RESISTANCE" of the whole section—

$$M = \frac{s \cdot d^2 \cdot b}{6}$$

If the cross-section of the beam is not a solid rectangle, but of the form shown in Fig. 19, the resistance of the parts omitted from the rectangle must be deducted.

The expression for the moment of resistance to be deducted will be the same in form as the above, but the maximum stress on this part—the base of the triangle—will be less in the ratio of d' to d, and the moment will be—

$$M = s \times \frac{d_1^2 \cdot b_1}{6} \times \frac{d_1}{d} = \frac{s \cdot d_1^3 \cdot b_1}{6\,d}$$

and the moment of resistance of the flanged section will be

$$M = \frac{s \cdot d^2 \cdot b}{6} - \frac{s \cdot d_1^3 \cdot b_1}{6\,d} = \frac{s\,(d^3\,b - d_1^3\,b_1)}{6\,d}$$

If the section is more complicated, but kept symmetrical, this formula will merely require to be extended; thus, if the depths and breadths of the absent parts are d_1, d_2, d_3, - - - d_n and b_1, b_2, b_3 - - - b_n, the expression for the moment of resistance will be—

$$M = \frac{s}{6\,d}\left\{d^3\,b - (d_1^3\,b_1 + d_2^3\,b_2 + d_3^3\,b^3 + \ldots . + d^3_n\,b_n)\right\}$$

This is the general expression for any symmetrical rect-

angular section. There is a factor involved in this formula which is in very common use in connection with stresses, which is called the "MOMENT OF INERTIA." The term is not expressive, as inertia means simply inaction, and a moment is connected with a force. I will, however, endeavour to make clear what the expression really signifies.

The triangle $h\, e'\, f^3$, Fig. 18, supplies the means of determining ratios of forces at different distances from the neutral axis if any are in action, and the distances themselves give the factors for calculating the moments of such forces; there remains the direct resistance of the material: this will be left out of the question for the present.

Assuming the factor to give a stress tension to be unity when $y = 1$, then the moment of a square unit will be $1 \times y^2$, because both factors vary as y. The sum of the areas will be $= \frac{d}{2} \times b$, and the mean moment of the whole area will be in the ratio of $\frac{d}{2} \times \frac{1}{2} = \frac{d}{4}$. The centre of gravity of these moments is $\frac{2}{3} \times \frac{d}{2}$ from the neutral axis, so the moment of inertia of the half section is—

$$I = \frac{d}{2} \times b \times \frac{d}{4} \times \frac{2}{3} \times \frac{d}{2} = \frac{d^3\, b}{24}$$

and for the whole section $I = \frac{d^3\, b}{12}$

If this factor is multiplied by the direct resistance of the material at unity from the neutral axis, the product should be equal to the moment of resistance of the section. If $s =$ the maximum resistance per square inch at the outside of the girder, then the resistance at unity from the neutral axis will be,

$$= s \times \frac{1}{\frac{1}{2}\, d}$$

and the moment of resistance is,

$$M = s \times \frac{1}{\frac{1}{2}d} \times I = s \times \frac{1}{\frac{1}{2}d} \times \frac{d^3 b}{12} = \frac{s \,.\, d^2 \,.\, b \,.}{6}$$

which is the same result as that previously reached.

The moment of inertia may be compared with a measure containing a certain number of cubic inches; in the former different results appear in the products according to the strength of the material under consideration, and in the latter different weights are obtained by filling the measure with liquids of different specific gravities.

If the flanges $A\,B$ and $C\,D$ are very thin in proportion to the depth, and the vertical web is also thin in proportion to the breadth, a simpler formula may be used, for the flange may be taken as a single layer. Let $t =$ the thickness of each flange and $t' =$ thickness of the web. The direct resistance of the flange is $= s \times b \times t$, and the moment of resistance is, for both flanges,

$$M = 2 \times s \times b \times t \times \frac{d}{2} = s \,.\, b \,.\, t \,.\, d$$

The web is neglected in dealing with the horizontal stresses, and regarded as carrying the vertical or shearing efforts. In applying this formula it is usual to measure the depth of the girder between the flanges, and not over all, or even to the centres of gravity of the flange areas, as in that case under a full load the maximum working stress would be exceeded in the outside layers. An example calculated both ways will be interesting as showing how closely the approximate method approaches the exact.

Fig. 20 represents a cross section of a built-up steel girder; each flange consists of two plates 18 inches wide and $\frac{1}{2}$ inch thick, and these are connected to a web plate

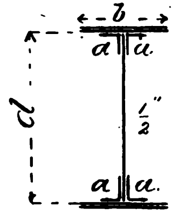


Fig. 20.

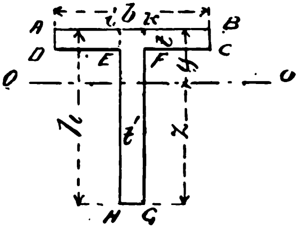


Fig. 21.

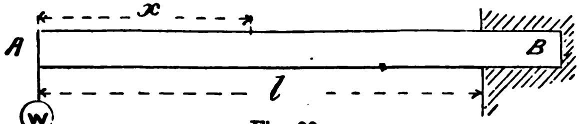


Fig. 22.

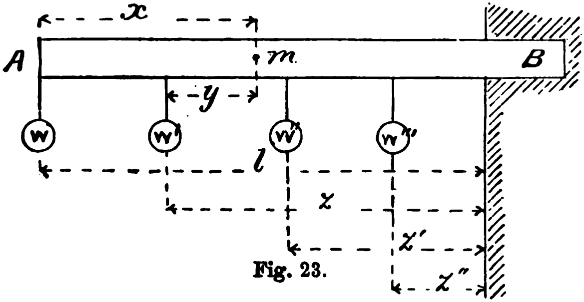


Fig. 23.

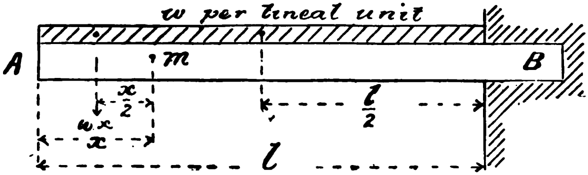


Fig. 24.

½ inch thick by angle steels *a*, *a*, *a*, *a*, each of which measures four inches on each limb, and is ½ inch thick. The horizontal limbs of the angle steels are regarded as forming part of the constructional area of flange, the vertical limbs serving as means of attachment to the web; the girder is 36 inches deep between the flange plates, and the maximum working stress for both flanges is taken as 7 tons per sectional square inch; this is assuming that the ultimate strength of the metal is not less than 28 tons, so that the factor of safety is not less than 4.

Working out the moment of resistance of the section by the first formula, we find,

$$M = \frac{s\,\{d^3 \cdot b - (d_1^3 \cdot b_1 + d_2^3 \cdot b_2 + d_3^3 \cdot b_3)\}}{6 \times d}$$

$d = 38$ ins., $b = 18$ ins. The parts to be deducted are three rectangles on each side of the web; adding each pair together we find, $d_1 = 36$ inches, $b_1 = 18 - (4 + \frac{1}{2} + 4) = 9{\cdot}5$ inches, deducting the widths of the angle steels and thickness of the web from the total breadth. $d_2 = 36 - 1 = 35$ ins.; by deducting the thicknesses of the angle irons, $b_2 = 7$ inches, the sum of the width of the angle steels in the clear. The remaining rectangles are those between the vertical limbs of the angle steels; therefore $d_3 = 36 - 2 \times 4 = 28$ ins.; $b_3 = 1$ inch.

Inserting the values in the above equation,

$$M = \frac{7\,\{\overline{38}^3 \times 18 - (\overline{36}^3 \times 9{\cdot}5 + \overline{35}^3 \times 7 + \overline{28}^3 \times 1)\}}{6 \times 38}$$

$$= 6827{\cdot}62 \text{ inch tons.}$$

course the resistance by the next formula, where the is left out, will give a much lower moment of resist-

ance. $b.t$ represents the sectional area of the flange, but $b'.t'$ must be added for the horizontal limbs of the angle steels; then

$$M = s.d.\{b.t. + b^1 t^1\} = 7 \times 36\ \{18 \times 1 + 8 \times 0{\cdot}5\}$$
$$= 6300 \text{ inch tons.}$$

In lattice and triangular girders the webs are not continuous, and would therefore not come into the first formula, which would then give,

$$M = \frac{7\ \{\overline{38}^3 \times 18 - (\overline{36}^3 \times 10 + \overline{35}^3 \times 7 + \overline{28}^3 \times 1)\}}{6 \times 38}$$
$$= 6111{\cdot}43 \text{ inch tons.}$$

When the section of the beam under bending stress is not symmetrical a different course is to be pursued. Fig. 21 shows a tee section, of which $A\ B\ C\ D$ is the table and $E\ F\ G\ H$ the web. The line $o \ldots o$ passes through the centre of gravity of the section, and therefore indicates the position of the neutral axis. The centre of gravity is found as follows. Assume the section to have a weight equal unity per square inch, and determine its moment of gravity about the edge $A\ B$, by multiplying the area of each part by the distance of its centre of gravity from $A\ B$, and adding the products together, then the sum divided by the total area will give the required distance y of the centre of gravity of the whole section from $A\ B$. There are three parts—the rectangle $i\ G$ and the two rectangles $D\ i$ and $K\ c$. As these component parts are in themselves symmetrical, the centre of gravity in each will also be the centre of the figure; then if t and t' are the thicknesses in inches of the table and web,

For rectangle $i\,G - h \times t^1 \times \frac{h}{2}$ = moment of gravity.

For rectangles $D\,i$ and $K\,c$, $(b - t^1) \times t \times \frac{t}{2}$ = moment of gravity.

Let the height $h = 6$ inches, breadth $b = 3$ inches, and $t = \frac{3}{4}$ inch, and $t' = \frac{1}{2}$ inch. Then, putting these values for the letters,

$$6 \times 0{\cdot}5 \times \frac{6}{2} = 9{\cdot}000$$

$$(3 - 0{\cdot}5) \times 0{\cdot}75 \times \frac{0{\cdot}75}{2} = 0{\cdot}703$$

$$9{\cdot}703 = \text{total moment of gravity}$$

The area of the section is $(6 \times 0{\cdot}5) + (2{\cdot}5 \times 0{\cdot}75) = 4{\cdot}875$ square inches.

$$y = \frac{9{\cdot}703}{4{\cdot}875} = 1{\cdot}99 \text{ inches,}$$

which in practice may be taken as two inches. We can now calculate the moments of inertia of the parts above and below the neutral axis. $z = 6 - 2 = 4$ inches.
For the upper part of the section,

$$I = \frac{d^3 \,.\, b - d_1{}^3 \,.\, b_1}{24} = \frac{2^3 \times 3 - \overline{1{\cdot}25}^3 \times 2{\cdot}5}{24} = 0{\cdot}799$$

For the lower part of the section,

$$I = \frac{d^3 \,.\, t'}{24} = \frac{4^3 \times 0{\cdot}5}{24} = 1{\cdot}33$$

For the whole section,

$$I = 0{\cdot}799 + 1{\cdot}333' = 2{\cdot}132$$

To find the moment of resistance this must be multiplied

by the resistance per square inch at one inch from the neutral axis. The maximum stress (7 tons) will be at HG, four inches from $o..o$; therefore the stress required will be $7 \div 4 = 1{\cdot}75$ tons per square inch, and the working moment of resistance of the whole section will be

$$M = 2{\cdot}132 \times 1{\cdot}75 = 3{\cdot}731 \text{ inch tons.}$$

The same stress is taken for compression as for tension, for so long as the elasticity of the material remains uninjured, equal extensions and compressions should follow equal pressures.

For purposes of calculation in connection with elasticity, deflection, and the like, a factor which is termed the "MODULUS OF ELASTICITY" is used. This quantity is for any substance the force which would—were such a thing possible—stretch a bar one inch square to twice its normal length; it is of course calculated from extensions of small range. The force at which the elongations begin to be irregular is known as the limit of elasticity; the ratio of this limit to the ultimate strength of the material varies widely in different materials. Cast iron holds its elasticity nearly to its breaking point, on account, no doubt, of its crystalline structure; in wrought iron the elastic limit is about 25 per cent. of the ultimate strength, and in mild steel it varies from 40 to 60 per cent. of the breaking stress.

It may be concluded from these figures that we are not to be guided solely by the breaking loads of materials as to the factors of safety to be used, for it is obvious that the limit of elasticity must not be approached; if the ratio of the limit of elasticity to ultimate strength is one third, the factor of safety should not be less than four, then the working stress will not be greater than 75 per cent. of

the limiting elastic stress, but when the elastic limit reaches half the ultimate strength, the factor of safety may be three; so, for instance, a lower factor of safety may be used with mild steel than that required for wrought iron.

Having investigated the nature of internal resistance to stress, I will now pass to the consideration of the forces accruing from external loads. In any case where the structure is at rest the moments of external force must, of course, be equal and opposite to the moments of resistance.

CANTILEVERS are beams firmly imbedded in a supporting wall at one end, and free at the other end, as shown in Figs. 22, 23, and 24. The useful load may be concentrated at the free end, as shown in Fig. 22, or divided into a number of local loads as shown in Fig. 23, or it may be distributed uniformly along the length of the cantilever as shown in Fig. 24. Every beam necessarily supports its own weight in addition to the superimposed load, but this is disregarded for the present, as its effect can be added subsequently, for if a girder supports a number of differently arranged loads, each one can be dealt with separately, and their moments of stress at any point added together to give the total or resultant moment at that point. In the three diagrams under consideration $A\,B$ is the cantilever of which the end A is free, and the end B firmly fixed. In Fig. 22 $W =$ the load at the free end; the moment of stress at any point distant x from the free end will be

$$M = W \times x$$

the maximum moment will occur at the face of the supporting wall, and will be

$$M = W \times l$$

In Fig. 23 there are shown four local loads, W, which may be equal to each other, or of different intensities. With the value of x indicated on the diagram two of these loads will be acting, and the moment of stress about the point m will be

$$M = W \,.\, x + W' \,.\, y$$

The maximum moment of stress at the face of the supporting wall will be the sum of the moments of all the vertical loads, and then will

$$M = W \,.\, l + W' \,.\, z + W'' \,.\, z' + W''' \,.\, z''$$

Fig. 24 shows a cantilever supporting an equally distributed load of w per lineal unit over its whole length. The moment of stress is required at a point m, distant x from the free end of the cantilever. The amount of load producing the moment of stress about the point m will be that included between that point and the free end of the cantilever, that is, $w \times x$, which being symmetrical will have its centre of gravity in its centre of length, and therefore distant $\frac{x}{2}$ from the point m; this load may be considered as concentrated in its centre of gravity, and therefore the moment of stress at the point m will be

$$M = w \,.\, x \times \frac{x}{2} = \frac{w \,.\, x^2}{2}$$

Similarly, to find the maximum moment which occurs at the point of support, the whole load, $w \,.\, l$, is regarded as concentrated at the middle of the length and the moment of maximum stress,

$$M = w \,.\, l \,.\, \times \frac{l}{2} = \frac{w \,.\, l^2}{2}$$

Girders freely supported at both ends, rest upon bearings near their extremities, which, however, are free to move; four cases of different loading on such are shown in diagram in Figs. 25, 26, 27, and 28.

In Fig. 25 $A\ B$ represents a girder carrying a central load W. This load being equidistant from the points of support A and B, the pressure upon each will be $W \div 2$, which is thus proved; the moment of the weight W acting vertically downwards about the support A is

$$M = W \times \frac{l}{2}$$

This will bring a pressure upon the support B, which must be equalised by an upward reaction R. The moment of this reaction about A will be

$$M = R' \times l$$

As these two moments will be equal when the stresses are in equilibrium,

$$\frac{W \,.\, l}{2} = R' \,.\, l\,; \text{ therefore } R' = \frac{W}{2}$$

The other half of the weight must rest upon the support A, and its reaction will therefore be

$$R = \frac{W}{2}$$

The stresses will under these circumstances necessarily be symmetrical upon each side of the centre of the span, so it will be sufficient to deal with one half of the girder. Let m show the position of a point in the neutral surface of the girder, about which the moment of stress is to be determined; its distance from the point of support A is x. The reaction R acts upwards, and therefore—to distinguish

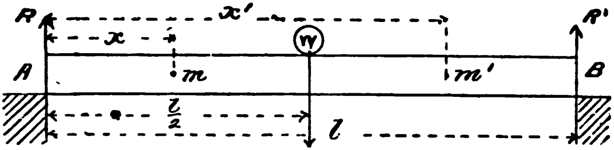


Fig. 25.

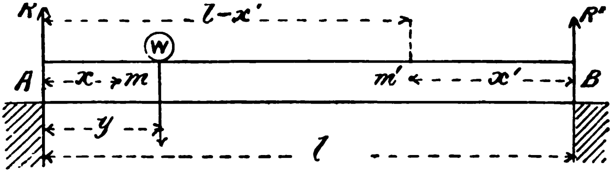


Fig. 26.

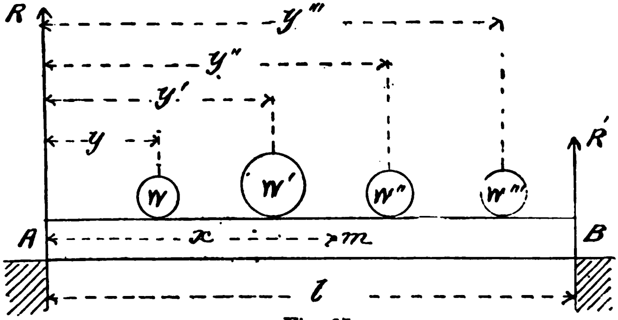


Fig. 27.

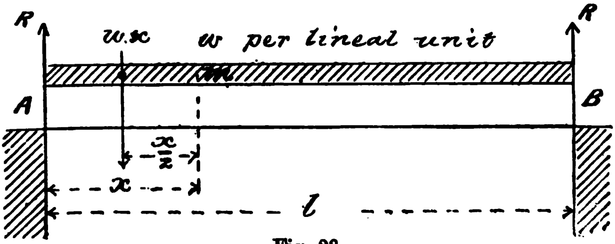


Fig. 28.

it from downward acting forces—it is given the negative sign, and

$$M = -R \times x = -\frac{W}{2} \times x$$

When the point at which the stress is required is at the centre of the span $x = \frac{l}{2}$ and

$$M = -\frac{W}{2} \times x = -\frac{W}{2} \times \frac{l}{2} = -\frac{W.l}{4}$$

I will here point out that wherever the resultant moment has a $-$ sign, the deflection of the girder is such that the upper surface is concave and the under surface convex; when the moment has a $+$ sign, the top of the girder is convex and the under side concave. In every case the concave surface is under compressive stress, and the convex surface under tensile stress.

Let the moment of stress be required at the point m', beyond the load W and distant x' from the support A. Working out this moment from both supports, a check upon the accuracy of the method is supplied.

Starting from A there will be two moments about the point m' — the upward negative moment of the reaction R acting at the distance x', and the positive downward moment of the load W acting at the distance $x' - \frac{l}{2}$ from the point m'. This moment will be

$$M = -R.x' + W.\left\{x' - \frac{l}{2}\right\}$$

$$= -\frac{W.x'}{2} + W.x' - \frac{Wl}{2} = \frac{W.x'}{2} - \frac{Wl}{2}$$

Starting from B, there will be one negative moment, R'

acting at the distance $(l-x')$ from the point m', and the moment will therefore be

$$M = -R' \times (l - x') = -\frac{W}{2} \times (l - x')$$

$$= -\frac{W.l}{2} + \frac{W.x'}{2}$$

the same equation as obtained above, but the component expressions are in different order.

The diagram Fig. 26 shows the girder $A\ B$ under a concentrated load at a point distant y from the support A. The moment of the load W about the support R will be

$$= W \times y$$

and the moment of the reaction of the support B about the same point—

$$= R' \times l$$

therefore,

$$W.y = R'.l; \text{ and } R' = \frac{W.y}{l}$$

Similarly, taking the moments about B,

$$W \times \{l - y\} = R.l; \text{ and } R = \frac{W.\{l - y\}}{l}$$

At the point m, distant x from A, the moment of stress will be

$$M = -R.x = -\frac{W\{l-y\}}{l}.x$$

At m', distant x' from the support B, the moment of stress will be

$$M = -R'.x' = -\frac{W.y}{l}.x'$$

If this result is calculated from the forces acting between

m' and A, two moments of force will be involved; the upward negative reaction R, and the positive downward pressure of the load W; the distance at which the former acts is $= l - x'$, and that at which the latter acts will be $= l - x' - y$; the resultant moment will therefore be—

$$M = - R\,\{l - x'\} + W.\,\{l - x' - y\}$$
$$= - \frac{W.\,\{l - y\}}{l} \times \left\{l - x'\right\} + W\left\{l - x' - y\right\}$$
$$= - W.l + W.y + W\,x' - \frac{W\,x'\,y}{l} + W.l - W.x' - W.y$$
$$= - \frac{W.x'\,y}{l}$$

which is the same value as that previously found.

The maximum moment of stress will occur immediately under the load W; calculated from the support A, it is,

$$M = - R.\,y = - \frac{W\,\{l - y\}}{l} \times y = - W.y + \frac{W\,y^2}{l}$$

Calculated from B, it is,

$$M = - R'\,\{l - y\} = - \frac{W.\,y}{l}\,\{l - y\}$$
$$= - W.\,y + \frac{W.\,y^2}{l}. -$$

The diagram Fig. 27 shows the girder AB loaded with several concentrated loads. Before the moment of stress at any point can be ascertained the reactions at the points A and B of support must be determined. Taking the moments of load and reaction about the point A, the following equation is found:

$$W.\,y + W'.\,y' + W''.\,y'' + W'''.\,y''' = R'.\,l$$

therefore, $R' = \dfrac{W.y + W' y' + W'' y'' + W'''.y'''}{l}$

and, $R = W + W' + W''. + W''' + R'.$

If the moment of stress is required at the point m, which in this example is placed between the second and third local load, it will be seen that three forces are concerned—the upward negative force of the reaction R, and the downward positive pressures of the loads W and W'. These forces give the following moments: $- R.x$; $+ W.(x - y)$; and $+ W'.(x - y')$, so that at the section at m

$$M = - R.x + W(x + y) + W'(x - y')$$

The resultant moment of stress immediately under—say $-W''$, will be,

$$M = - R.y'' + W.(y'' - y) + W'.(y'' - y') + W''.(y''' - y'')$$

Figure 28 represents the girder $A\ B$ carrying an equally distributed continuous load of w per lineal unit. The total load is $w \times l$, and as it is equally distributed along the girder, one half will be carried by each end support; therefore $R = R' = -\dfrac{w.l}{2}$

The moment of stress at a point m distant x from the point of support A will be the difference between the upward negative moment of the reaction R and the positive moment of the load lying between A and m. The distance at which R acts about the point m is x, therefore the negative moment at m is

$$= R \times x = -\frac{w.l.x}{2}$$

The load between A and $m = w \times x$, and its centre of

gravity—where it may be regarded as concentrated—is at the centre of its length, and therefore $\frac{x}{2}$ distant from the point m and its moment is,

$$= w . x \times \frac{x}{2} = \frac{w . x^2}{2}$$

The resultant moment of stress will therefore be,

$$M = \frac{w . x^2}{2} - \frac{w . l . x}{2} = \frac{w}{2}\left\{x^2 - l . x\right\}$$

This expression may be verified by calculating the stress from the support B, the distance of m from which is $l - x$: the distance of the centre of gravity of the load between m and B will be $\frac{l-x}{2}$, and its weight is $w\,\{l - x\}$

The negative moment will be,

$$= - R' \times \{l - x\} = - \frac{w . l}{2} \times \{l - x\} = - \frac{w . l^2}{2} + \frac{w . l . x}{2}$$

The positive moment will be,

$$= w \times \{l - x\} \times \frac{l - x}{2} = \frac{w . l^2}{2} - \frac{2\,wl . x}{2} + \frac{w . x^2}{2}$$

The resultant moment will be,

$$M = \frac{w . l^2}{2} - w . l\,x + \frac{w . x^2}{2} - \frac{w . l^2}{2} + \frac{w . l . x}{2}$$

$$= \frac{w . x^2}{2} - \frac{w . l . x}{2}$$

the same as before.

By means of this equation the moment of stress can be

determined for any section in the length of the girder, and the sections can be modified to suit these varying stresses. How this is done will be shown when dealing with the practical application of the formula, in Chapter IV.

When the girder is of such manufacture that it holds the same section throughout its length—as in the rolled iron and steel joists now so largely used in building construction, that section must be sufficient to support the maximum stress on the girder.

It is now necessary to ascertain the position of the point of maximum stress upon the girder. The moment of stress is nothing when $x = o$, therefore it increases from one point of support until it reaches a maximum, and thence decreases until it becomes nothing at the opposite point of support. In between these points lies that at which the maximum moment of stress occurs. Let x be the distance of this point from the support A; on each side of this maximum the moment will be less, so if two equations are taken, giving x in one a small increment, h, and in the other a diminution, h, these equations will be equal to each other, and the value of x corresponding to a maximum stress will be found. Using the form $\frac{w}{2}\ \{x^2 - l.\,x\}$ we have,

$$\frac{w}{2}\{(x+h)^2 - l\,(x+h)\} = \frac{w}{2}\ \{(x-h)^2 - l\,(x-h)\}$$

therefore,

$$x^2 + 2\,h\,x + h^2 - l.\,x - l.h = x^2 - 2\,h.\,x + h^2 - l.\,x + l\,h$$

Cancelling equal quantities on both sides of the equation, there remain,

$$2\,h\,x - l.\,h = -\ 2\,h.\,x + l\,.\,h$$

Transposing, and dividing both sides by h,

$$4x = 2l$$

and, $$x = \frac{l}{2}$$

Therefore the maximum moment occurs when $x = \frac{l}{2}$ when,

$$M = \frac{w}{2}\left\{x^2 - l.x\right\} = \frac{w}{2}\left\{\frac{l^2}{4} - l \times \frac{l}{2}\right\} = -\frac{w.l^2}{8}$$

GIRDERS WITH FIXED ENDS.—*A B*, Fig. 29, is an exaggerated elevation of a girder with free ends under a uniformly distributed load; *C* and *D* are the points of support. The effect of the load in deflecting the central part of the girder is to cause its extremities to rise up from their supports as shown. It is now necessary to ascertain how the stresses will be affected if the ends of the girder are firmly fixed as shown in Fig. 30; this may most easily be explained by assuming that a girder with free ends having been brought into a condition of deflection, has its ends forced down upon the supports in the position they occupied before any load came upon the girder.

While the ends of the girder remain free the upper surface is in compression throughout its length, and the lower surface is in tension throughout; when the ends are pressed down the upper surface is drawn partly into tension and the under surface is partly compressed, and a moment of stress set up immediately over the supports where previously there was no stress.

The moment of stress on the free girder is a varying quantity, and it is obvious that the moment set up in restoring the ends of the girder to their original position must

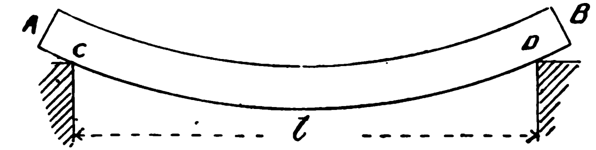


Fig. 29.

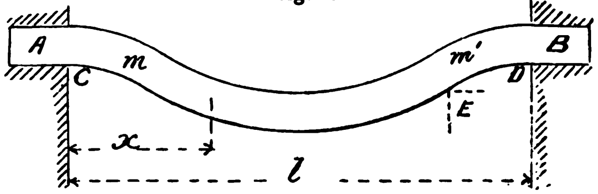


Fig. 30.

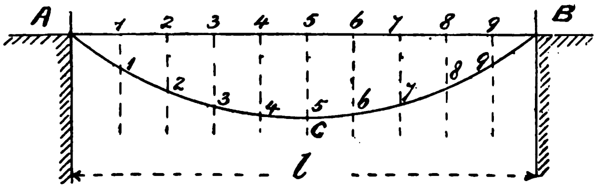


Fig. 31.

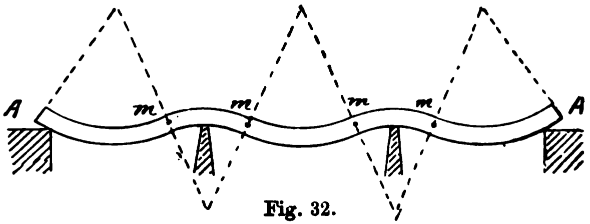


Fig. 32.

equal the average or mean moment of stress in the free girder.

By calculating the moments of stress at various points 1, 2, 3, &c., Fig. 31, along the span *A B*, and plotting them as ordinates 11, 22, &c., a number of points are found through which a curve of moments may be drawn, as shown by the line *A C B*. The area enclosed between the straight line *A B* and the curve *A C B* represents the sum of all the moments of stress upon the girder, and therefore the mean ordinate will represent the mean moment of stress. By reference to any text-book on conic sections we find that the equation to the moments of stress is also an equation to a parabola, and the mean ordinate of a parabola is two thirds of the maximum ordinate. The maximum ordinate represents the maximum moment of stress, therefore the mean ordinate, which equals the moment of stress over either point of support, will be—

$$M' = \frac{w \cdot l^2}{8} \times \frac{2}{3} = \frac{w \cdot l^2}{12}$$

and this moment is positive, as it is produced by the downward pressure upon the ends of the girder; for some distance at each end the girder will therefore be convex upon the upper surface and concave on the under side.

The equation for the moment of stress on a fixed girder, at a point distant x from the point of support, will be—

$$M = \frac{w \cdot l^2}{12} + \frac{w \cdot x^2}{2} - \frac{w \cdot l \cdot x}{2}$$

$$= \frac{w}{12}\left\{l^2 + 6\,x^2 - 6\,x\right\}$$

As the ends of the girder are subject to stresses opposite in character to those obtaining in the central part, there must

be two points in the span at which there are no moments of stress, and it is here that the curvature changes; wherefore these points are termed the points of contrary flexure, or more shortly, the "points of contra-flexure," shown at m and m'.

To find the value of x corresponding to the point of contra-flexure, the equation to the moment of stress must be made equal to O; then,

$$M = O = \frac{w}{12}\left\{l^2 + 6x^2 - 6lx\right\}$$

$$\frac{wx^2}{2} - \frac{wlx}{2} = \frac{wl^2}{12}$$

The factor w occurring on both sides of the equation can be eliminated, leaving an adfected quadratic equation,

$$x^2 - lx = -\frac{l^2}{6}$$

Completing the square, we find,

$$x^2 - lx + \frac{l^2}{4} = \frac{l^2}{4} - \frac{l^2}{6} = \frac{l^2}{12}$$

and extracting the square roots of each side,

$$x - \frac{l}{2} = \pm \frac{l}{\sqrt{12}}$$

$$x = \frac{l}{2} \pm \frac{l}{\sqrt{12}} = l\,\{0{\cdot}5 \pm 0{\cdot}291\}$$

This gives two values for x, one for each point of contra-flexure; they are,

$$x = 0{\cdot}791\,l, \text{ or } 0{\cdot}209\,l$$

A girder, fixed at one end only, and free at the other

would be in the case of the part $A\ m\ m'$ of the girder shown, and its one point of contra-flexure would be at m, and the distance of this point from C, in terms of the span, will be—

$$\frac{0 \cdot 209}{0 \cdot 791} \cdot l = 0 \cdot 264\ l$$

The dotted lines at E indicate a pier supporting the free end of the girder, fixed at A. In Fig. 32 is shown a continuous girder of three spans; when this is equally loaded throughout its length, the centre span is in the position of a girder fixed at both ends, and each end span is similarly circumstanced to a girder fixed at one end and free at the other, but the spans at each end should be to the centre span as 0·791 is to 1.

A greater number of spans between the end ones will also be as girders with fixed ends.

It will not, however, always happen that the bridge is loaded over its entire length, so this has to be considered in each case, and the safest plan is to calculate the stresses from the dead weight of the structure as for a continuous girder, and those from the live load as for single spans, as one span may be covered with live load, whilst there is none on the adjoining spans. The points m, m, &c., on the diagram, are those of contra-flexure under a uniformly distributed load. A fixed girder, a central span of a continuous girder, consists virtually of two cantilevers with a free girder supported at their ends.

TRIANGULAR OR OPEN-WEBBED GIRDERS are composed of bars so arranged that the loads bring direct stresses upon them in tension and compression, but no bending stresses. Fig. 33 shows the simplest form of triangular girder; the bars A 1; 1, 7; 7, 2; 2, 8; 8, 3; 3, 9; 9, 4; 4, 10; 10, 5;

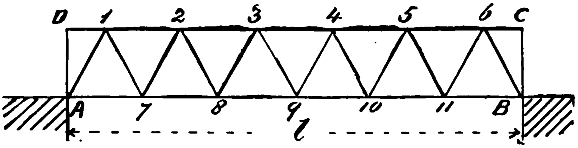


Fig. 33.

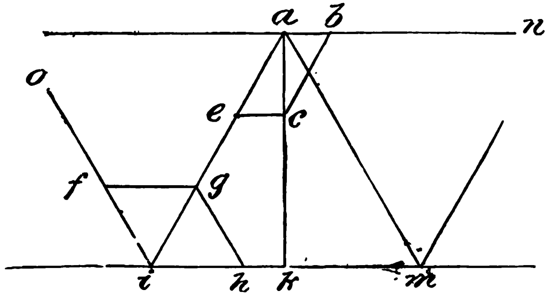


Fig. 34.

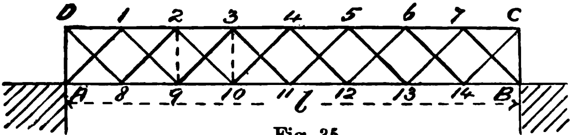


Fig. 35.

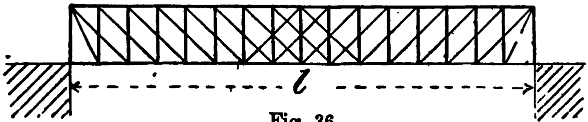


Fig. 36.

5, 11; 11, 6; and 6 *B*, form the web, and the bars 1 to 6 and *A* to *B* constitute the top and bottom flanges. The parts *A D* 1 and *B C* 6 merely complete the rectangular form of the girder; the triangles are equilateral. The load is carried by the web bars to the piers *A* and *B*, and these bars are kept in position by the horizontal flanges. Let there be a uniformly distributed load carried on the top of the girder; this may then be regarded as concentrated at the apices, 1, 2, 3, 4, 5, 6, of the triangles forming the girder. On each apex, 2, 3, 4, 5, there will be a load equal to the weight per lineal foot multiplied by the distance between two consecutive apices; let W represent this load; then on the end apices, 1 and 6, the load will be 0·75 W, because half of the load between *D* 1 and *C* 6 will be carried by the end struts, *D A* and *C B*.

It is evident that a load resting on the top of a web bar will bring compression upon that bar, and a load or stress coming on the bottom of a bar will put tension upon it. It will happen that certain tensions and compressions act upon one bar, and then the difference between their sums will be the resultant stress. I will now show the relation of load to stress on these bars.

In Fig. 34, *a i m* is one triangle of the web of the girder; let the proportion of the weight, W, at *a*, which is carried by the bar *a i*, be represented by the vertical line *a c*, complete the parallelogram *a b c e*, then will *a e* represent the thrust upon the bar, *a i* and *a b*, that brought upon the top flange at the point *a*; this will have to be added to stress—if any—brought upon the flange at other points to the left of *a*, to give the total stress on the bar *a n*; similarly the stress *a e* will be added to other stresses—if any—acting at the point *a* from the bar *m a*. The stress on *a i* is taken up by the bars *i o* and *i m*. From the point *i* on the line *i a* mark off *i g*

equal to the total stress on $a i$, and complete the parallelogram $i f g h$; then $i f$ will represent tension brought on the bar $i o$ at the point i, and $i h$ that accruing at the same point on the bar $i m$. The stress passing along $i o$ will be similarly resolved at the top upon the top flange and the next web bar.

It will be seen that as the sides of the parallelograms either coincide with the centre lines of the elements of the structure, or lie parallel to them, those centre lines may be used for finding the factors of stress obtained from the measurements of the work itself, for, by similar triangles,

$$\frac{a e}{a c} = \frac{a i}{a k}; \text{ and } \frac{i h}{i g} = \frac{i m}{i a}$$

But in these expressions $a k$ is the effective depth of the girder, $a i$ = the length of a lattice bar, and $i m$ = the base of a triangle. $a b$ and $e c$, being opposite sides of a parallelogram, are equal, and

$$\frac{e c}{a c} = \frac{k i}{a k}$$

$k i$ = half the base of a triangle, because the line $a k$ is drawn vertically, and the base is horizontal. Let L = the length of a web bar between the points where its centre line intersects the centre lines of the top and bottom flanges; let D = the depth from the intersection a to the centre line of the bottom flange; and let B = the base of a triangle between two successive intersections of web bars with the flange centre lines: then—

$$L = a i; \; B = i m = 2 i k; \text{ and } D = a k$$

Inserting these values in the above expressions we find

$$\frac{a e}{a c} = \frac{L}{D}; \; \frac{i h}{i g} = \frac{B}{L}; \text{ and } \frac{e c}{a e} = \frac{B}{2 D}$$

Let W = the vertical load at a, and S = total stress upon the web bar $a\,i$. Then the stresses due to the load W will be—

$$\text{Thrust on web bar} = W \times \frac{L}{D}$$

$$\text{Thrust on top flange} = W \times \frac{B}{2\,D}$$

The stresses at the foot of the web bar will be

$$\text{Tension on next web bar} = S \times \frac{f\,i}{i\,g} = S \times \frac{a\,i}{a\,m} = S$$

because the lengths of all the lattice bars are equal.

$$\text{Tension on bottom flange} = S \times \frac{B}{L}$$

As a rule, the angles of the struts and ties with the horizontal numbers is either 60°, as shown in Fig. 33; 45°, as shown in Fig. 35; or the struts vertical and the ties at an angle of 45°, as shown in Fig. 36. This last form has the advantage of keeping the struts to a minimum length, which, as their resistance diminishes with the increase of the ratio of length to least width, effects a saving, while the length of a tie does not affect its tensile resistance.

If the angle is 60°, the three sides will be equal, so B will be $= L$, and by the properties of right-angled triangles:

$$D = \sqrt{L^2 - \left(\frac{L}{2}\right)^2} = \sqrt{\frac{3\,L^2}{4}} = 0{\cdot}866\,L$$

therefore,

$$\frac{L}{D} = \frac{L}{0{\cdot}866\,L} = 1{\cdot}154\,; \quad \frac{B}{L} = 1\,; \quad \frac{B}{2\,D} = \frac{L}{1{\cdot}732\,L} = 0.577$$

If the angle of the web bars with the horizontal is 45°, as

shown in Fig. 35, each web bar will be the diagonal of a square, of which the sides are formed by the depths 2, 9, and 3, 10, for instance; and two half-bases, 2, 3, and 9, 10. Therefore the half-base is equal to the depth, and the length of a web bar is

$$L = \sqrt{2\,D^2} = 1{\cdot}414\,D$$

therefore,

$$\frac{L}{D} = \frac{1{\cdot}414\,D}{D} = 1{\cdot}414\,;\ \frac{B}{L} = \frac{2\,.\,D}{1{\cdot}414\,D} = 1{\cdot}414\,;\ \frac{B}{2\,D} = 1$$

In the arrangement shown in Fig. 36 the length of the struts is equal to the depth D, and that of the ties $= 1{\cdot}414\ D$ and $B = D$.

The methods of summarising the stresses on each bar will be shown in the practical examples of different types of bridge girders, which will be treated in detail in subsequent chapters, in which also some examples of trussed bridges will be considered, the investigation by the parallelogram of forces being applicable to all such cases.

CHAPTER II.

THE THEORY OF STRESS AND RESISTANCE IN ARCHES AND SUSPENSION BRIDGES.

IN a properly formed upright arch the stresses are all compressive, and in an inverted arch they are tensile; but for this condition of things to obtain, the form of the arch must be such that the line of stress shall keep well within its outer and inner contours, which are termed respectively the extrados and intrados or soffit. As a general rule, under a maximum load the line of stress should not pass above or below the central third of the depth of the arch. When the load per lineal foot of span is given, and the stress at the crown has been determined, it is a simple matter to set out the line of stress. I shall first consider an upright arch, of which one half is shown in elevation in the diagram, Fig. 37. $A\,B$ is the half arch, which is supported at A by an abutment, and at the crown B meets the other half of the arch.

When the arch is uniformly loaded over its whole span the thrust at the crown will be horizontal, and its value may be determined by the principle of moments of force. Let $w =$ the load in tons per lineal foot of span; $R =$ the vertical load on the abutment, and, therefore, its reaction upwards; $l =$ effective span, and $v =$ effective rise or

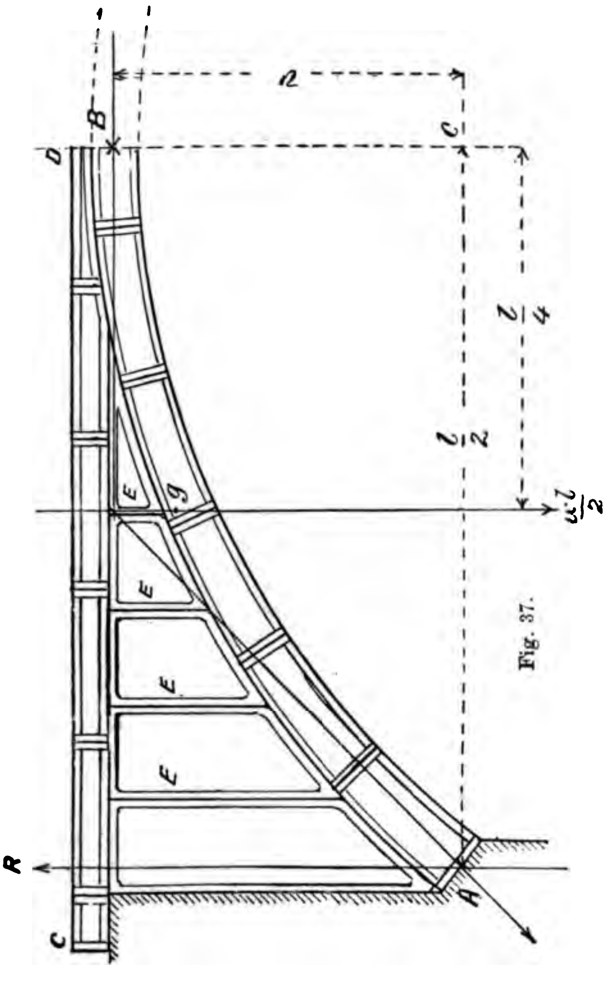


Fig. 37.

versed sine of the arch, both in feet; these will, in fact, be the span and rise of the line of thrust. Let T = the horizontal thrust in tons at the crown, and T_1 = the thrust upon any other section of the arch normal to the line of stress. The arch being uniformly loaded, one half of its total weight will be the vertical load on each abutment; therefore,

$$R = \frac{w \cdot l}{2}$$

and this force acts upwards, about the point c—where the versed sine meets the chord of the arc—at a horizontal distance $= \frac{l}{2}$.

The weight of half the arch may be considered as concentrated at its centre of gravity, g, which, if the load is uniformly distributed, will be midway between the crown of the arch and its abutment; that is, at a horizontal distance from the point $c = \frac{l}{4}$.

This force acts downwards, so the resultant moment of stress about the point c will be

$$M = \frac{w \cdot l}{2} \times \frac{l}{2} - \frac{w \cdot l}{2} \times \frac{l}{4} = \frac{w \cdot l^2}{8}$$

The moment of resistance about the point c is obviously the horizontal resistance at the crown of the arch multiplied by the rise v, and when equilibrium occurs the thrust must equal the resistance; therefore, the moment of resistance will be

$$M = T \times v$$

The moments of resistance and stress being equal, it follows that

$$T \times v = \frac{w \,.\, l^2}{8}$$

whence

$$T = \frac{w \,.\, l^2}{8\,v}$$

In cases where the load is not uniformly distributed—as, for instance, where the dead weight of the structure increases as the abutments are approached—the position of the centre of gravity g will be altered and the resultant moment of stress with it, so that under such conditions the horizontal thrust is modified. It is generally necessary to increase the sectional area of the arch towards the abutment and the spandril; filling between the road girder and the arch must necessarily become deeper in the same direction; but when the arch is of iron or steel the difference is not sufficient in comparison with the total load to materially affect the practical calculations, so the load taken for the determination of these may be taken as uniformly distributed, except in some unusual cases.

Under this assumption I shall now proceed to show the method of determining the proper contour of the arch.

The diagram, Fig. 38, shows the outline of half an arch of which the effective span is 42 feet, and the rise 8 feet. The depth of the rib is taken as 2 feet, and the load 1 ton per lineal foot of span. The thrust at the crown will be,

$$T = \frac{w \,.\, l^2}{8\,v} = \frac{1 \times \overline{42}^2}{8 \times 8} = 27{\cdot}562 \text{ tons.}$$

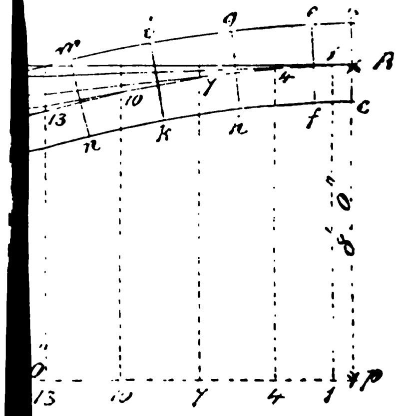
A is the centre of the crown and *B* the abutment of the arched rib *a b c d*, and for the purpose of calculation the load is divided up into segments *b e f c*, *e g h f*, *g i k h*, *i m n k*, and so

on until the abutment is reached. The load on the ment is 1 ton, that on each of the following ones Through the centre of the crown draw a horizonta meeting at o a vertical line $B\,o$ drawn upward centre B of the abutment, and from this point B dr horizontal line $B\,p$, meeting in the point p a ver $b\,c\,p$ drawn through the crown of the arch; then sents the effective semi-span, and $A\,p$ the rise sine of the arch.

By finding the resultants of horizontal thrust dead loads between the crown and the abutment, th thrusts and the contour of the line of thrust will mined. It will not be necessary to give the whole ogram, two contiguous sides being sufficient for sought, and with the resultant making up a tri forces.

The load on each segment is regarded as concen its centre of gravity, 1, 4, 7, 10, &c., and acting in tion of the vertical lines 1—1, 4—4, 7—7, &c.

On the line $A\,o$, and from the point 1, mark off a 1—2, representing to scale the horizontal thrus tons; from the point 2 draw a vertical line 2—3, e ton, the load on segment $b\,f$; join 1—3, then 1—3 e resultant thrust passing through the point 4, and a its direction. Produce 1—3 to 5, making 4—5 = from the point 5 let fall a vertical line 5—6, equal the load upon segment $e\,h$; join 4—6, which is the stress passing through the point 7. Produce 4—6 making 7—8 = 4—6, and from point 8 let fall line 8—9 equal 2 tons, the load upon segment $g\,k$, 7—9, for the resultant passing through the point continue describing these triangles of force until *ment is reached*, and then the curve of thrust can



[*To face p.* 64.

through the points A, 1, 4, 7, 10, 13, 14, to 19 and B, and this curve should not pass outside the middle third of the depth of the rib; in the diagram it is well within it, but irregularities of moving load will cause some displacement.

As the triangles of force are all right-angled triangles, we find a simple rule for determining the thrust at any point distant x from the crown of the arch. Let T_1 = the thrust required, then

$$T_1 = \sqrt{T^2 + (w \cdot x)^2} = \sqrt{\left(\frac{w\,l^2}{8\,v}\right)^2 + (w \cdot x)^2}$$

Let the stress at the abutment be required, $x = 21$,

$$T_1 = \sqrt{\left(\frac{1 \times \overline{42}^2}{8 \times 8}\right)^2 + (1 \times 21)^2} = \sqrt{1200{\cdot}66}$$
$$= 34{\cdot}65 \text{ tons.}$$

If the working resistance of the metal is taken for steel as 6 tons, then the sectional area of the rib at the crown must be $27{\cdot}562 \div 6 = 4{\cdot}59$ square inches for every ton of load per lineal foot, and at the abutment $34{\cdot}65 \div 6 = 5{\cdot}75$ square inches per ton of load per lineal foot.

Steel or iron arches will usually occur in sets of not less than two, and they must be braced together every 4 or 5 feet if they are not to be treated as columns.

A useful, practical rule for wrought-iron and mild-steel struts and columns which are built up of plates is one evolved by Professor Gordon, and is as follows:—Let r = the length of the strut divided by its least width or diameter, f = factor of safety used, and s the working resistance to compression per sectional square inch, then

$$S = \frac{19}{f} \div \left\{1 + \frac{r^2}{900}\right\}$$

A considerable number of expressions for the resistance of struts have been published, but I have found the above quite satisfactory for practical purposes.

Many arch-shaped bridges are not arches in principle, but consist of two end cantilevers with a straight girder carried between them, such, for instance, as Westminster Bridge, which has cast-iron cantilevers springing off each side of each pier and bolted together over the pier; to the ends of the cantilevers wrought-iron girders are bolted.

In the ordinary arched bridge the road or railway supported by it rests upon a straight girder, as shown by *C D* (Fig. 37). This straight girder will be horizontal and supported towards the abutments by struts *E*, which fill in the spandril and divide the road girder into very short spans. Each strut will be calculated by means of the formula just given, and the road girder will be treated as a continuous girder.

Another arrangement known as a tied arch has come into very common use for railway bridges; the thrust from the haunches, instead of being met by the resistance of the abutments, is taken up by a horizontal tie which connects the opposite ends of the arched ribs. Fig. 39 shows an elevation of a tied arch in which *A B C* is the arched rib, and *A D C* the horizontal tie. The roadway is supported by cross girders fixed at the lower ends of suspension members, *e*, *e*, *e*, &c., for which reason these are called suspension bridges by some people. The bottoms of the suspension members are kept in position by the main tie, and distortion under unequal loading is resisted by counter-bracing, *f*. The main tie *D* is in tension only, and should therefore be made quite straight. I mention this particularly because the mistake of giving it an upward camber has been made, which resulted in the rupture of the counter-bracing as soon as the

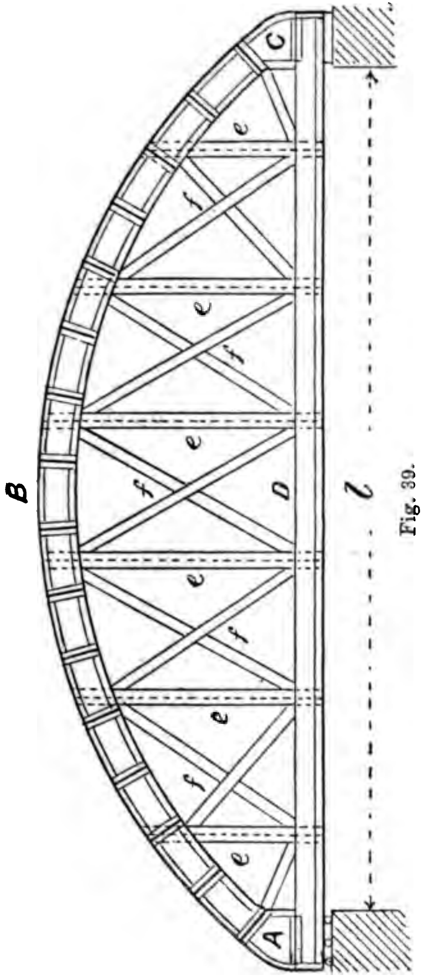


Fig. 39.

thrust of the loaded arch came into action. The tension of the tie is constant throughout its length, and is equal to the thrust at the crown in intensity, as will be obvious if we take the moments of force about a point in the crown of the arch. One end *A* of the arch rests upon rollers to allow of alterations of length under varying temperatures.

This form of structure must not be confounded with the round-back triangular-webbed girder which has its web formed of struts and ties, and which is in principle a trussed girder, whereas the tied arch has no web, but only connecting members to convey the load to it; the stress on the uprights *e*, *e*, &c., and on the counter-bracing *f* is simply tension, but it is advisable to make the uprights of a stiff section to oppose any tendency to lateral oscillation. With this form of bridge the only stresses put upon the supports are vertical—there is no overturning thrust to be dealt with.

The commonly recognised form of suspension bridge is shown in elevation at Fig. 40. *A B C D E* is a continuous chain running over saddles in towers, *G G*, and anchored at each end behind a mass of buried masonry. Vertical suspension rods carry the floor *F F F* of the bridge. The stresses upon the suspension chain are in amount the same as those on an arch of the same proportions, but they are tensile instead of compressive. The chain may be run over a number of consecutive spans if required before being anchored.

The light and elegant appearance of this form of suspension bridge greatly recommends it, but, and especially in the earlier examples, it is liable to great faults, of which its liability to oscillation is the most serious. Although a broken or mixed load, such as a mob of people crossing such a bridge, will not put it in motion, synchronized movements, as, for instance, the marching of soldiers, will set up a dangerous amount of lateral oscillation.

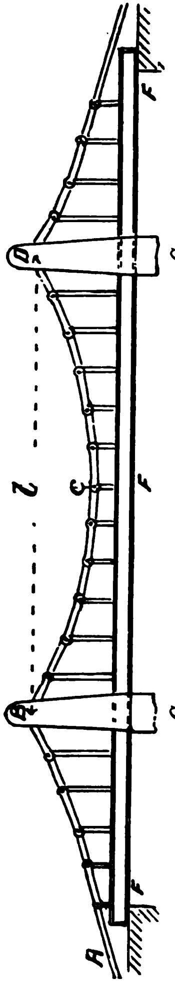


Fig. 40.

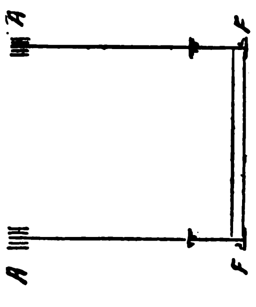


Fig. 41.

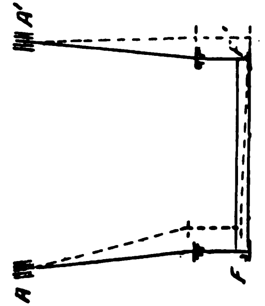


Fig. 42.

The form of cross section adopted in the earlier suspension bridges is shown in Fig. 41; *A A* are the main chains, *F F* the flooring, and *A F* the suspension rods carrying the flooring. These suspension rods being vertical and therefore parallel, will, if set in oscillation, move synchronously, because they are equal in length and parallel, and the weight of the floor will be all that tends to bring them to rest; the chains will also take up the vibration, so that the main chains and suspension rods can be caused to vibrate like parallel pendulums, and if the impulses causing such vibration happen to be isochronous with the normal period of oscillation of the chain and suspension rods, this vibration may increase until the structure is endangered.

By arranging the chains and suspension rods in inclined positions, as shown in Fig. 42, an element of stability is introduced, because there will be initial tension on the suspension rods in addition to that due to the direct floor load, and if oscillation occurs, displacing the suspension rods as shown by the dotted lines, the initial stress will be diminished on the rod $A^1 F^1$, and increased on the rod *A F*, and this will tend to bring the floor back to its normal position.

In some bridges the suspension rods have been inclined away from the centre of the span upwards towards the towers, to reduce the longitudinal undulation of the floor, and it has also been proposed to counter-brace the suspension rods, but this would probably complicate the stresses without doing much good in other directions. The parapet girder to which the floor is connected should be made sufficiently rigid to distribute concentrated loads over several suspension rods.

CHAPTER III.

BRIDGE LOADS AND THE CLASSIFICATION OF BRIDGE GIRDERS.

UPON the accurate determination of the loads to which a bridge may be subject will depend, on one hand, the safety of the structure, and on the other economy of material.

The student should never forget that in dealing with the strength of materials, or with the loads and stresses to which they may be subjected, no greater folly can be committed than to work upon averages. Results are published showing the average strength of different brands of iron or steel; these are useless, for it is only upon the minimum strength that reliance can be placed; in the same way it is of no use to know the average loads on bridges: each bridge must be strong enough to support the heaviest load which can ever come upon it.

A crowd of people will not weigh more than 120 lbs. per superficial foot, and that may be accepted as the maximum live load for foot bridges, but when we come to vehicular, no maximum can be fixed for practical use, as its application would lead to waste of metal, each case must be considered by itself.

If a railway bridge is required, it must be designed—in England at least—to meet satisfactorily the Government test, a train of locomotive engines of the heaviest type used on the railway. To ascertain the loads on the bridge, a horizontal line must be drawn representing the span, and on

this the positions of the wheels, with the load on each, are to be marked. At each point will be a concentrated load, and to find the maximum moment of stress, these loads must be dealt with as shown in Chapter I. The concentrated loads in such cases vary very much, as for instance, from ten tons on a pair of tender-wheels to sixteen tons on a pair of driving-wheels on the engine. Now, it is obvious that it would be no use to average these loads and call them uniformly distributed.

Road bridges for general traffic are liable to infinite varieties of loads. In country districts such heavy machines as steam ploughs may pass over them, and in towns steam road rollers, some of which weigh over thirty tons. In these cases the loads are necessarily concentrated upon the wheels of the vehicle, and these concentrated loads call for the closest attention in designing the details of a bridge.

The main girders of a bridge support the flooring, and any point in this flooring—in a bridge for general traffic—may become the point of application of a concentrated load; it is therefore imperative that each subsidiary girder, which forms part of the floor, shall be designed to support such concentrated load on any part of its length, and that the flooring between these girders shall also be strong enough to support it.

A bridge, to be properly designed, must have the stresses calculated, beginning at the roadway, and passing through all the subsidiary girders to the main girders which carry the whole work.

The floor may consist of small transverse or cross girders, supported at each end by the main girders, on either the top or bottom flanges; when the flooring is on the top of the main girders, it is often spoken of as decking. If the main girders are to be made with plate webs, the cross

girders will be put about 4 feet apart, and the intermediate space filled up with buckled or other stiffened plates.

When the main girders have triangular webs, it is necessary to connect the cross girders with the flanges at the points of junction of the struts and ties with the flanges, so that no transverse stress shall be brought upon the flanges, and in large structures the distances between these points will be too great for filling in with plates, and therefore, intermediate longitudinal girders will be required between the cross girders, and the filling-in plates will be carried by the intermediate girders.

There is another kind of longitudinal girder used when the cross girders are not far apart, and its duty is to distribute concentrated loads over several cross girders. As an instance, let the cross girders be 4 feet apart, and subject to the passage of a heavy vehicle which has its wheels 8 feet apart longitudinally; when the front and back wheels are on two cross girders, there will be one in between which receives no load at all, and those beyond the wheels will also be unloaded.

The two loaded cross girders will deflect under the loads which come upon them, and if all the cross girders are connected by a light longitudinal girder, riveted above or beneath them, the otherwise idle girders will receive a part of the load, for the intermediate girder must deflect with those on each side of it. These longitudinal girders are termed "distributing girders," and do not carry any part of the load to the piers. In railway bridges, their use, when carefully designed, saves about one-third of the weight of the cross girders. The forms of bridge floors most generally in use may be better understood by reference to the illustrations, Figs. 43 to 50.

Fig. 43 shows a longitudinal section of part of a bridge

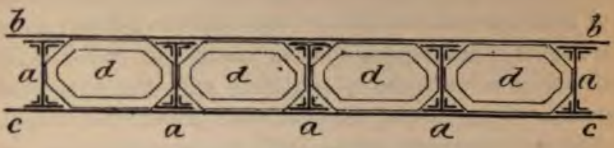


Fig. 43.

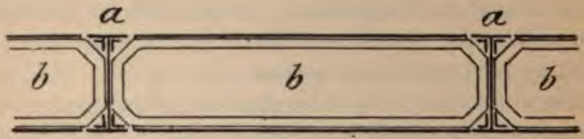


Fig. 44.

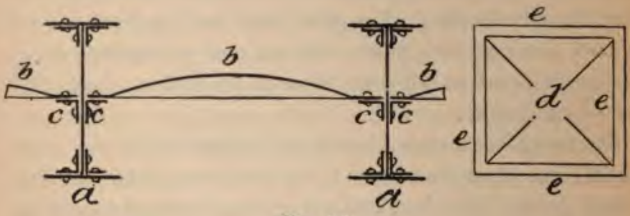


Fig. 45.

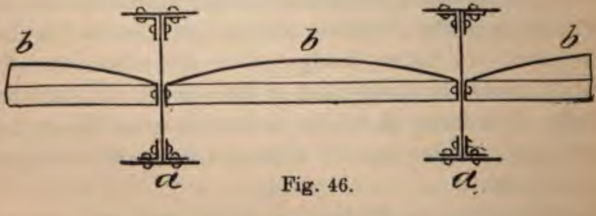


Fig. 46.

floor composed of cross girders about 4 or 5 feet apart, and indicated by the letters *a a a a a*. These are connected by a distributing girder, which is so arranged as not to project either above or below the floor; the top flange *b b* and the bottom flange *c c* are each made continuous, running respectively over the top flanges, and under the bottom flanges of the cross girders. The web of the distributing girder is made up of plates *d d d d* with angle-iron frames on both sides, the return limbs of the angle-irons are riveted to the top and bottom flanges of the distributing girder, and their ends are riveted together through the webs of the cross girders. I have found this construction give a very rigid floor in practice for railway bridges.

Fig. 44 shows a longitudinal section of the flooring of a triangular webbed girder, in which the cross girders *a a* are placed at greater distances apart, and intermediate longitudinal girders *b* carried between them. It will be seen in this arrangement, the girders *b* are independent of each other, and their flanges stop against those of the cross girders at each end. The web is connected to the flanges by angle-iron frames on each side, and the ends are riveted together through the webs of the cross girders.

A light and useful floor, which has been largely used for both railway and road bridges, is shown in longitudinal section in Fig. 45. The cross girders *a* are placed about 4 feet apart, and the space between filled in with buckled plates *b*, connected to the webs of the cross girders by means of angle-irons *c c*. A buckled plate is shown in plan at *d*. There is a flat fillet *e e e e* all round its margin, but the central part is dished up between dies, so that it has a rise at the crown, which for a 4-feet buckled plate, ¼ inch thick, would be about 3 inches. The angle-irons *c c*, are riveted through their vertical limbs to the webs of the cross girders

and the buckled plates are riveted—or bolted—to the horizontal limbs of the angle-irons *c c*.

It may not always be desirable to have the buckled plates in the position shown, which is especially suitable for railway bridges, with the sleepers bedded between the cross girders. In other circumstances they may rest upon and be riveted to either the top or bottom flanges of the cross girders, but they should always be fixed with the convex side upwards, as then the fillet is in tension, if they are inverted the fillet will be in compression—a stress which a thin strip of metal is not suited to sustain. It will be seen from the illustration that three rows of rivets are requisite for the connection of the buckled plates at each cross girder web, and to reduce the cost of this, I, some years back, designed buckled plates with vertical fillets, fitted between the cross girders, as shown in Fig. 46; by this arrangement only one row of rivets was necessary to make the connection with the cross girder webs, and the longitudinal joints between the buckled plates themselves were made with one row of rivets instead of the two rows of rivets necessary when tee-iron joint covers are used to connect the flat fillets. These plates, however, have one disadvantage, which is that they require to be most carefully made to fit properly between the cross girders. With the flat filleted buckled plates, they can be cut if any happen to be a little too large, and if any are a trifle small, the bearing angle-iron makes up for it. Mild steel plates can be bent or dished cold, but this should not be done when it is to be used for structural purposes in the form of buckled plates, for they will then have a tendency to resume their original flat condition.

In order to get rid of the complexity of girders and floor plates, several manufacturers have designed floorings of

various forms to combine the two, the first of these being brought out about twenty-five years since; it was of a trough form, but made of plates joined by angle-irons, so there was really nothing gained by its use.

A marked improvement on this was made by the introduction of troughs *a*, shown in Fig. 47. These troughs are each in one piece, and do not require connecting angle-irons. They are placed side by side and connected together by longitudinal cover strips *b* riveted on to them. This trough flooring is of one thickness throughout, and therefore metal is wasted in the webs where its moment of resistance is small, which could be used to much greater advantage in the flanges.

A saving in riveting was made subsequently by making each trough with one side lower than the other, as shown in Fig. 48; the flanges on the higher sides of the troughs overlap those on the lower sides of the next troughs and are riveted to them, thus saving the cover strip and one row of riveting, but still the equal thickness of flange and web remained. A great advance was made in trough flooring, or decking as it is sometimes termed, by the adoption of trough sections of the form shown in Fig. 49. Here the troughs *c* have thick bottoms *a a*, &c., and the sides *b b*, &c., are tapered away towards their edges. These troughs are placed together as shown, alternately upright and inverted, and the thin edges of the sides *b b*, &c., are riveted together by rivets *d d*, &c., which will run along the neutral axis. We have in this a trough flooring with thick flanges and thin webs, which is about as satisfactory as any that can be designed; all the rivets also are in shearing stress, to resist which they are best suited.

A light and shallow trough flooring about 4 in. deep is shown in Fig. 50; this has been much used in Indian and

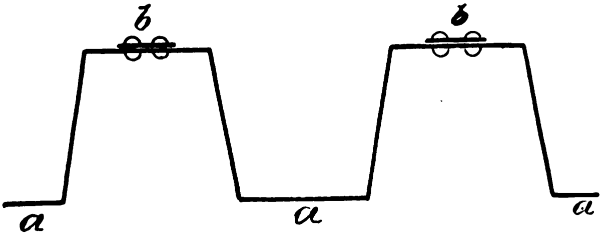


Fig. 47.

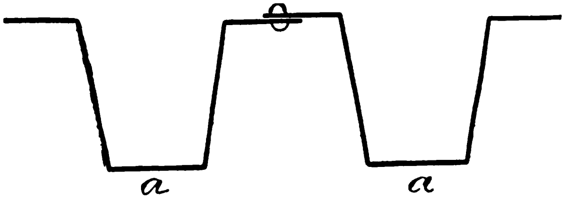


Fig. 48.

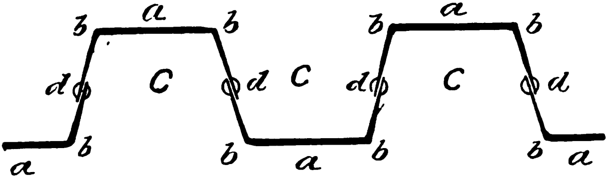


Fig. 49.

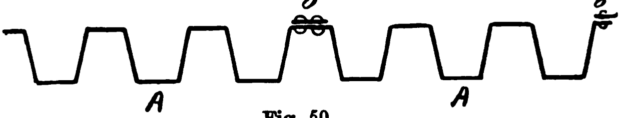


Fig. 50.

other railway bridges for filling in the flooring between the rail-bearing girders where no heavy load can come—unless a train becomes derailed, in which case the wheels would probably go through and leave the vehicles astride the floor girder. These plates have three or four troughs in one piece and may be joined together by cover strips *b b*.

There are several other kinds of decking, some consisting of semi-tubes riveted together side by side, and prevented from spreading by tie-bars, but I cannot say that I like floors which depend upon tie-bars to retain their normal widths, especially where simplicity of construction is sought.

The trough floors described above are not only used for the floors of long-span bridges, in which they transmit the loads to the main girders in the same way as the ordinary cross girders; but in short spans they are laid longitudinally across, and so serve as main girders and flooring together. The troughs are sometimes filled up with concrete to form a level surface, but this adds considerably to the dead weight of the structure.

The order of classification of girders is then: starting from the useful load, intermediate longitudinal girders; cross girders; main girders; if there are distributing girders, these come in as adjuncts to the cross girders; if trough flooring is used it takes the place of cross girders.

We have certain fixed data upon which to start; the live or useful load for any particular bridge can be ascertained and its maximum determined, also the weight of ballast, floor-plates, and other covering material which forms the roadway and is part of the dead weight; we then come to the first of the girders, and this, besides supporting the superincumbent load, has its own weight to carry. At one time a guess used to be made, and if on calculating

the weight it was found inaccurate, a second approximation taken and so on. This going over the calculations several times involved a considerable expenditure of time and labour, and I therefore determined, from economically designed girders, factors which would give closely the weight of a girder of known proportions to carry any given load, then the weight of the girder thus found is added to the superincumbent weight to get the total load, from which the final section of the girder can be at once calculated.

These factors are given for loads uniformly distributed over the girders; so if the loads are localised, a uniformly distributed load which would give the same maximum moment of stress, must be used with the factor to determine the weight of the girder.

The figures given in the following table are for iron, subject to working stresses of 4 tons per sectional square inch in compression and 5 tons in tension.

Let $W =$ weight of girder in tons per lineal foot of span, $W_2 =$ superincumbent load, including live or moving load and all dead load, except the weight of the girder itself; in tons; $c =$ a coefficient taken from the table. Then,

$$W = W_2 \times c.$$

The plate girders are divided into two classes; those having a uniform sectional area throughout; and those in which the flange plates are proportioned to the varying stresses along the span. The first column shows the ratio of the effective span to the depth of the girder; the second gives the coefficient of weight for uniform section of flanges; and the third column that for uniform stress per square inch upon flanges:—

Span. Depth.	Uniform Section of Flanges.	Uniform Stress on Flanges.
8	0·00182	0·00145
9	0·00198	0·00156
10	0·00213	0·00167
11	0·00228	0·00178
12	0·00243	0·00189
13	0·00259	0·00200
14	0·00274	0·00211
15	0·00289	0·00222
16	0·00304	0·00233

To illustrate the use of this table, let it be required to determine the weight of a girder 40-ft. span, to carry a load of 60 tons equally distributed over its length. The depth between the flanges is usually taken as the effective depth, let this equal 3 ft. 4 in., then the ratio of span to depth will be 12, and the factor for uniform stress on flanges is = 0·00189. Therefore—

$$W = W_2 \times c = 60 \times 0{\cdot}00189 = 0{\cdot}1134 \text{ ton per foot of span.}$$

The weight of the girder should therefore be—

$$= 0{\cdot}1134 \times 40 = 4{\cdot}536 \text{ tons.}$$

If the girder is to be of mild steel, it will be lighter in ratio to the higher working stress adopted; if the tensile resistance is taken as 6·5 tons, against the 5 tons for wrought iron, the weight of the girder will be—

$$4{\cdot}536 \times \frac{5}{6{\cdot}5} = 3{\cdot}49 \text{ tons.}$$

The factors for rigid arches and suspension chains are as follow—for iron.

Span. / Rise.	Rigid Arch.	Suspension Chain.
4	0·00085	0·00073
5	0·00093	0·00079
6	0·00100	0·00085
7	0·00108	0·00091
8	0·00117	0·00097
9	0·00125	0·00104
10	0·00133	0·00110
11	0·00141	0·00116
12	0·00149	0·00121

CHAPTER IV.

PRACTICAL APPLICATION OF FORMULÆ.

By equating the formulæ for stress with those giving the resistances in cases where the principle of moments obtains, we find the direct stresses and from them determine the sectional areas required. In those cases in which the stresses are found by the parallelogram of forces, the direct stresses are obtained, and no farther calculation is necessary, except to ascertain the sectional area necessary to meet such stress.

The ultimate strengths of the materials used are ascertained by testing bars, or pieces cut from bars or plates; these have not suffered any deterioration of strength or homogeneity by processes of manufacture, but the plates and angle and tee-bars used in the construction of a bridge girder undergo the vicissitudes of punching—or drilling—and some of them joggling under a press, or hammer, and for all this, some allowance must be made.

The improvements in the manufacture of steel have been so great that our leading engineers now use it with confidence, though so recently as twelve years since many plates were found unreliable, they would crack under variations of temperature.

It would be out of *place* to criticise here the different *manufactures of steel,* but I wish to impress upon the

younger readers of this work the necessity of conscientiously studying the qualities of material with which they in future practice may have to deal, for nothing is more wearying than a state of doubtfulness as to whether the material specified will be supplied according to the specification, unless it be the inability to judge or test it.

If a plate or bar is under tensile stress, any holes punched or drilled in it, will reduce its effective sectional area—the acting sectional area of a plate 12 inches wide and $\frac{1}{2}$-inch thick, with two rows of rivet-holes $\frac{3}{4}$-inch in diameter will be equal to $10{\cdot}5 \times \frac{1}{2} = 5{\cdot}25$ square inches—but if the stress is compressive, and the rivets or bolts fill the holes, they will not cause any loss of resisting area, as the body of the bolt or rivet will take up the pressure and pass it on; if, however, the bolt is not a solid fit in the hole, there is the same loss of area as would occur if the plate were in tension. In the case of punched holes the loss is rather more than that of the metal punched out, because the material is weakened for some little distance around the hole through the violent action of the punch; when the metal is drilled out there is no such injury to the metal, and, therefore, it is common practice to punch holes smaller than their required size and broach them out in a broaching machine, thus cutting away the damaged material. When a number of plates in tiers are to be riveted together they can be economically drilled in a multiple drilling machine, which will drill the holes through all the thicknesses of plates at one operation and thus ensure absolute coincidence of the holes in the successive tiers of plates. If the plates are punched separately there is a chance that the holes will not in all parts coincide, and then the rivets will be distorted in filling them, and their transverse sectional areas reduced; if the holes are intended for bolts

and are not lineable, they must be further broached out to get a true hole, and thus more material cut away, increasing the loss of effective section.

For bridges subject to considerable vibration, a factor of safety of five should be used; the Board of Trade is satisfied with something less, giving a fixed working stress of 5 tons per sectional square inch as the limit for both tension and compression, and a great deal of the girder iron used in Great Britain has not a higher ultimate stress than 22 tons per sectional square inch, though some brands run up to 25 or 26 tons.

Mild steel is specified to have an ultimate tensile strength between 28 and 32 tons per sectional square inch; higher resistance is accompanied by a tendency to brittleness; of course the calculations of sectional areas will be made upon the basis of the lower figure.

Let a plate girder be required to carry a load of 2 tons per lineal foot of span over an opening 120 feet wide. Let w = load per foot run; l = span; d = effective depth; A = sectional area of flange in square inches, the form being similar to that shown in Fig. 20.

The effective depth is to be measured between the top and bottom flanges and the effective span from centre to centre of the bed plates upon which the ends of the girder are supported. Let these bed plates be 5 feet long, then the effective span will be 125 feet. The depth must next be determined.

Let s = working resistance of the material in tons per square inch; a = the sum of the sectional areas of both flanges in square inches; a' = vertical sectional area of web in square inches including an allowance for the weight of web stiffeners; t = thickness of web in inches.

As the moment of resistance has for its leverage the depth of the girder, it is obvious that the deeper the girder is made, the less will be the sectional area of the flange required, so by increasing the depth we economize in the flanges. The vertical sectional area of the web will be theoretically constant, as only a direct shearing stress comes upon it, but in practice the plates cannot be used below a certain thickness, so that after a particular depth is reached any extension of it increases the weight of the web.

If then diminution of flange weight is accompanied by increase of web weight, there must be some ratio of depth to span in each case that will correspond to a minimum weight of the girder as a whole. The weights of the various parts will, of course, vary as their sectional areas. The moment of stress at any point distant x from one point of support has been shown to be,

$$M = \frac{w\,x^2}{2} - \frac{w\,l\,x}{2}$$

and the moment of resistance is the direct resistances of the two flanges multiplied by half the depth of the girder, and the moment of resistance is necessarily equal to the moment of stress, therefore,

$$M = a \times \frac{d}{2} \times s$$

therefore,

$$\frac{a\,.\,d\,.\,s}{2} = \frac{w\,x^2}{2} - \frac{w\,l\,x}{2}$$

$$a = \frac{w\,x^2}{d\,s} - \frac{w\,l\,x}{d\,s}$$

also, $a' = 12\,d\,t$; hence $a + a' = \frac{w\,x^2}{d\,s} - \frac{w\,l\,x}{d\,s} + 12\,d\,t.$ —

Here is found the positive quantity $\frac{w x^2}{d s} \times 12 d t$ and the negative $-\frac{wlx}{d s}$ varying with d; if, therefore, a point is reached at which the value $a + a'$ is a minimum and about to change from the decreasing to the increasing value, we may by making the increment of d infinitesimal, regard the increase of the two quantities as equal. Let f be an indefinitely small increment of d, then

$$a + a' = \frac{w x^2}{s(d+f)} + 12 t (d+f) + \frac{w l x}{s(d+f)}$$

Deducting these from the previous quantities, and equating the differences of the positive and negative values, we get

$$\frac{w x^2}{d s} - \frac{w x^2}{s(d+f)} + 12 t d - 12 t (d+f) = \frac{w l x}{s(d+f)} - \frac{w l x}{d s}$$

whence,

$$12 t f = \frac{w x^2}{s}\left(\frac{1}{d} - \frac{1}{d+f}\right) - \frac{w l x}{s}\left(\frac{1}{d+f} - \frac{1}{d}\right)$$

$$= \frac{w x^2}{s} \times \frac{f}{d^2 + d f} - \frac{w l x}{s} \times \frac{f}{d^2 + d f}$$

As f is taken infinitesimally small in comparison with d, the value $d f$ as compared with d^2 may be neglected, then dividing both sides of the equation by f we find

$$12 t = \frac{w x^2}{s . d^2} - \frac{w l x}{s . d^2}$$

Multiplying both sides of this equation by d,

$$12 t d = \frac{w x^2}{s d} - \frac{w l x}{s d^2} \text{ ; or } a = a'$$

Therefore, when the weight of the girder is a minimum, the sum of the sectional areas of the flanges will be equal to the area of the web.

The value of d at any point may be found from the equation $12\,t = \frac{w\,x^2}{s\,d^2} - \frac{w l x}{s\,d^2}$ thus

$$t = \frac{w\,x^2}{12\,s\,d^2} - \frac{w\,l\,x}{12\,s\,d^2}\,;\ \text{and}\ d^2 = \frac{w\,x}{12\,t\,s}\ (x - l)$$

$$d = \sqrt{\frac{w\,x}{12\,t\,s}\,(x - l)}$$

Changing the sign brings this to a rational quantity without altering the difference in value between x and l, and,

$$d = \sqrt{\frac{w\,x}{12\,t\,s}\,(l - x)}$$

but this is an equation to an ellipse, of which the length of the girder forms the major axis, and the depth the semi-minor axis. From this it appears that the most economical form for a plate girder, under an equally distributed load, is that of a semi-ellipse. In the example I am now taking, however, I shall take the flanges as parallel for convenience in some other respects. To find the proper depth at the centre, make $x = \frac{l}{2}$ then

$$d = \sqrt{\frac{w\,.\,l}{24\,t\,s} - \left(l - \frac{l}{2}\right)} = \sqrt{\frac{w\,.\,l^2}{48\,t\,s}}$$

The next step will be to find the thickness of web with allowance for stiffeners.

The shearing stress upon the web at any point distant

y from the centre of the span is $= w \,.\, y$; it is nothing at the centre of the span when the girder is fully loaded and is a maximum over the points of support. The amount of shearing stress at any point is that equal to the load transmitted through that point to the nearest point of support. If the girder is not fully loaded, there will be a shearing stress at the centre of the span, and this must be provided for. If the girder is loaded for half its length, this load will be $= \frac{w\,l}{2}$, and it may be considered as concentrated at one-fourth of the span from the support at which the load commences; then the load transmitted through the web to the farther support will be—

$$= \left(\frac{w\,l}{2} \times \frac{l}{4}\right) \div l = \frac{w\,l}{8}$$

and this will be the shearing stress on the web at the centre of the span. Over the support the maximum shearing stress will be half the total live load, plus half the weight of the girder itself.

The girder would not be made less in depth than one-twelfth of the span, for reasons which will appear when dealing with "Deflection," so this proportion is safe to take to determine the dead weight of the girder. Assuming that the plates in the flanges are to be reduced to accord with the diminishing stresses towards the points of support; the factor of weight for this proportion is, according to the table, page 81, 0·00189. – $W_2 = 2$ tons $\times$ 125 feet $= 250$ tons; therefore the weight of the girder in tons per lineal foot

$$= 250 \times 0{\cdot}00189 = 0{\cdot}472 \text{ tons,}$$

this brings the value of w up to $2 + 0{\cdot}472 = 2{\cdot}472$ tons

per lineal foot; therefore the shearing stress over the supports will be—

$$= 62 \cdot 5 \times 2 \cdot 472 = 154 \cdot 5 \text{ tons.}$$

The working stress for iron in shearing is 4 tons per sectional square inch, therefore the web area over the points of support should be $154 \cdot 5 \div 4 = 38 \cdot 625$ square inches. At a depth of one-twelfth the span, that is, $125 \div 12 = 10 \cdot 41\dot{6}$ feet, the theoretical thickness would be—the depth of web being $10 \cdot 41\dot{6} \times 12 = 125$ inches;

$$\frac{38 \cdot 625}{125} = 0 \cdot 309 \text{ inch,}$$

for a depth as great as this, I should not put in a web less than $\frac{1}{2}$-inch thick at the supports, and $\frac{3}{8}$-inch thick at the centre of the span, which would give an average web thickness of $\frac{7}{16}$-inch for the plates. The web stiffeners would be tee irons, 5 inches by 5 inches, by $\frac{3}{4}$-inch thick on both sides of the web-plates at every four feet along the girder, and these would also serve as covers to the joints. The horizontal sectional area of these tee-irons will be $= 2 \left\{ 5'' + 5'' - \frac{3}{4}'' \right\} \frac{3}{4} = 13 \cdot 875$ square inches, and this spread along 4 feet, that is, 48 inches, will correspond to an average thickness of

$$\frac{13 \cdot 875}{48} = 0 \cdot 289 \text{ inch.}$$

The value of t will, therefore, be $\frac{7}{16} \times 0 \cdot 289 = 0 \cdot 726$ inch. Let $s = 4$ tons. The proper depth of the girder will be—

$$d = \sqrt{\frac{w \,.\, l^2}{48\, t\, s}} = \sqrt{\frac{2 \cdot 472 \times \overline{125}^2}{48 \times 0 \cdot 726 \times 4}} = 16 \cdot 647 \text{ feet.}$$

This height would require heavier web stiffeners than I have assumed, and to avoid this additional weight I shall fix the height of the girder at 14 feet, and proceed with the calculations on that basis. The resistance of one flange is equal to its sectional area A multiplied by the working stress s per square inch, which is here taken as 4 tons for wrought iron. The moment of resistance at any point where $A =$ the sectional area of flange in square inches is—

$$M = s . A . d$$

Equating this with the moment of stress at any point distant x from one point of support—

$$s A d = \frac{w x^2}{2} - \frac{w l x}{2}$$

$$A = \frac{w x^2}{2 s d} - \frac{w l x}{2 . s . d} = \frac{w}{2 s d} \left\{ x^2 - l x \right\}$$

For compression s will be taken $=$ 4 tons, and for tension $s = 5$ tons per sectional square inch. By calculating the stresses at various points along the span, ordinates are found from which a curve, such as that shown in Fig. 31, page 51, can be drawn, and this will show how the sectional area of the flange should be reduced to suit it to the diminishing stress; there is, however, a shorter way of getting at this, and one more convenient in practice. Each flange of the girder will consist of one or more plates connected to the web by angle-irons on each side; if one plate is sufficient for the central stress, there can be no reduction of sectional area, except by making the flange of several lengths of plates of different thicknesses, and this would not be worth while as cover-joint plates would take up what is saved in the main plates. If, however, two or more

tiers of plates are required at the centre of the span, none but the inside ones of each flange need be run the whole length of the girder, the others may be made shorter according to the diminution of stress.

To return to the example, the area at the centre is given, when $x = \frac{l}{2}$; then—

$$A = \frac{w}{2 \, . \, s \, . \, d} \left\{ \frac{l^2}{4} - \frac{l^2}{2} \right\} \frac{w \, . \, l^2}{8 \, . \, s \, . \, d}$$

$$= \frac{2 \cdot 472 \times \overline{125}^2}{8 \times 4 \times 14} = 86 \cdot 217 \text{ square inches}$$

for the top flange; and for the bottom flange—

$$A' = \frac{2 \cdot 472 \times \overline{125}^2}{8 \times 5 \times 14} = 68 \cdot 974 \text{ square inches.}$$

For the top flange this gives the gross sectional area, and the net area for the bottom flange. The flange plates will be 30 inches wide, and the angle-irons connecting them with the webs 6 inches × 6 inches × $\frac{5}{8}$ thick, with two rows of rivets in each limb; the rivets to be $\frac{7}{8}$ inches in diameter, and all rivet-holes are to be drilled from the solid, or broached out from $\frac{5}{8}$-inch punched holes.

Beginning now with the top flange—its gross area is to be, not under 86·217 square inches. The horizontal limbs only of the angle-irons are to be taken in as flange area, the amount will be $= 2 \times 6 \times \frac{5}{8} = 7\frac{1}{2}$ square inches. The remaining area to be made up in plates, will be $= 86 \cdot 217 - 7 \cdot 5 = 78 \cdot 717$ square inches. The thickness of the plates at the centre will be, altogether, $78 \cdot 717 \div 30 = 2 \cdot 623$ inches, which, to suit the commercial thicknesses of the plates, will be taken as $2\frac{5}{8}$ inches.

It is evident that the more plates in the thickness the more nearly can they be cut off to correspond to the curve of stress; therefore I shall make up this central thickness with four tiers of plates, ½ inch thick, and the outside tier ⅝ inch thick.

One of the equations given above determines the required area at any given point, and by transposing this we can find the position of a point to suit a given area. As we cannot practically taper the thicknesses of the plates, the flange area is reduced by jumps stopping off one plate at a time.

In the case in question the stoppage of the ⅝-inch plate will reduce the flange area to $86{\cdot}25 - \{30 \times 0{\cdot}625\} = 67{\cdot}5$ square inches, and each stoppage of ½-inch plates will further reduce the flange area by 15 square inches, successively bringing it down to 52·5, 37·5, and 22·5 square inches.

The expression for the value of x for a given sectional area is thus found :—

$$A = \frac{w\,x^2}{2\,s\,d} - \frac{w\,l\,x}{2\,s\,d}$$

therefore,

$$x^2 - l\,x = \frac{2\,s\,.\,A\,.\,d}{w}$$

This is an adfected quadratic equation solved thus : complete the square on the left-hand side in the usual way, making an equal addition to the other side to preserve the equality of the two sides, then—

$$x^2 - l\,x + \left(\frac{l}{2}\right)^2 = \left(\frac{l}{2}\right)^2 - \frac{2\,s\,A\,d}{w}$$

In this transposition the minus sign is given to $\frac{2\,s\,A\,d}{w}$

because the solution of $x^2 - l\,x$ must give this sign to th result, l being always greater than x. Taking the squar roots of both sides of the equation we have—

$$x - \frac{l}{2} = \sqrt{\frac{l^4}{4} - \frac{2\,s.A.d}{w}}$$

$$x = = \frac{l}{2} \pm \sqrt{\frac{l^2}{4} - \frac{2\,s\,A\,d}{w}}$$

It is evident from the form of the equation th $\sqrt{\frac{l^2}{4} - \frac{2\,s.A\,d}{w}}$ is equal to the distance which the pla extends on each side of the centre of the span, as t quantity is to be either added to $\frac{l}{2}$ or deducted from it, giving the two values of x, one for the commencement a the other for the termination of the tier of plates; the fore the length of the tier of plates will be,

$$= 2\sqrt{\frac{l^2}{4} - \frac{2\,s.A\,d}{w}}$$

For the example under consideration this formulæ become

$$2\sqrt{\frac{l^2}{4} - \frac{2\,s\,A\,d}{w}} = 2\sqrt{\frac{\overline{125}^2}{4} - \frac{2 \times 4 \times A \times 14}{2{\cdot}472}} =$$

$$2\sqrt{3906{\cdot}25 - 45{\cdot}307\,A}$$

When the outer tier of plates—⅝ inch thick—stops, th sectional area of the top flange is reduced to 67·5 squar inches, so the length of this outer tier,

$$= 2\sqrt{3906{\cdot}25 - 45{\cdot}307 \times 67{\cdot}5} = 58{\cdot}24 \text{ feet.}$$

The stoppage of the next plate reduces the sectional are

of the flange to 52·5 square inches, so the length of this tier will be,

$$= 2 \sqrt{3906{\cdot}25 - 45{\cdot}307 \times 52{\cdot}5} = 78{\cdot}16 \text{ feet},$$

and the two following tiers will have lengths respectively,

$$= 2 \sqrt{3906{\cdot}25 - 45{\cdot}307 \times 37{\cdot}5} = 93{\cdot}96 \text{ feet},$$

and,

$$= 2 \sqrt{3906{\cdot}25 - 45{\cdot}307 \times 22{\cdot}5} = 111{\cdot}11 \text{ feet}.$$

The lengths of the tiers actually used will be the nearest above these to suit the pitch—or distance from centre to centre—of the rivets, for, of course, each tier of plates and each plate in the tier must comprise a certain number of pitches.

If a rivet occurs in the centre of the flange there will be an odd number of pitches, but if there is a space there this number will be even. In the latter case, the first tier with rivets 4 inches pitch would be, 60 feet = 180 pitches; the second tier, 79 feet 4 inches = 238 pitches; the third, 94 feet = 282 pitches; and the fourth, 112 feet = 336 pitches. The inside tier of plates runs the whole length of the flange.

The nett sectional area required at the centre of the bottom flange—which is in tension—has been found to equal 68·974, say 69 square inches. The rivets in the limbs of the angle-irons will be zig-zagged so as not to take more than one rivet-hole out of each limb in a cross section square to the length of the angle-iron: the sectional area available to resist tension in the horizontal limbs of the two angle-irons will be $= 2\,\{6 - \frac{7}{8}\}\,\frac{5}{8} = 6{\cdot}4$ square inches. Deducting this it leaves sectional area to be supplied by the plates $= 69 - 6{\cdot}4 = 62{\cdot}6$ square inches.

The bottom flange will be the same width as the t 30 inches, but out of the section four rivet-holes are to taken, for it will be necessary to rivet the edges of plates together to prevent the entrance of moisture betw them, by which internal corrosion would be set up; effective width of these flange plates will, therefore, $30 - \frac{7}{8} \times 4 = 26{\cdot}5$ inches. The thickness of the bot flange at the centre of the span must be $62{\cdot}6 \div 26{\cdot}$ 2·324 inches—commercially, $2\frac{3}{8}$ inches. An outside p $\frac{3}{8}$ inch thick, and four inner ones, each $\frac{1}{2}$ inch thick, furnish the requisite sectional area at the centre. reduction of area by stopping the outside plate will $= 26{\cdot}5 \times \frac{3}{8} = 9{\cdot}937$ square inches; this will le $69 - 9{\cdot}937 = 59{\cdot}063$ square inches, say 59. The stopp of each of the succeeding $\frac{1}{2}$-inch plates will cause a red tion of $26{\cdot}5 \times \frac{1}{2} = 13{\cdot}25$ square inches, and this will red the sectional area of the flange successively to 45·75 squ inches; 32·5 square inches and 19·25 square inches. actual sections will be somewhat more, because the s commercially obtainable are larger than the theoret thickness, but this will be neglected, for sometimes weights run a little under the nominal ones.

The outside tier of plates will have a length,

$$= 2\sqrt{\overline{\frac{125}{4}}^2 - \frac{2 \times 5 \times 59 \times 14}{2{\cdot}472}}$$

$$= 2\sqrt{3906{\cdot}25 - 52{\cdot}54 \times 59} = 56{\cdot}78 \text{ feet.}$$

The succeeding tiers of plates will, in consecutive or have the foll[illegible]ths:—

$$2\text{nd tier}, = 2\sqrt{3906{\cdot}25 - 52{\cdot}54 \times 45{\cdot}75} = 77{\cdot}52 \text{ feet};$$

$$3\text{rd tier}, = 2\sqrt{3906{\cdot}25 - 52{\cdot}54 \times 32{\cdot}5} = 92{\cdot}98 \quad ,,$$

$$4\text{th tier}, = 2\sqrt{3906{\cdot}25 - 52{\cdot}54 \times 19{\cdot}25} = 109{\cdot}44 \quad ,,$$

The actual lengths will, of course, be regulated to suit the pitch of the rivets the same as in the top flange.

Whatever particular duty a plate-girder may be taking, whether cross or main, or bearing girder, the method of calculating the constituent areas will be the same, so the above example fully shows the application of the formula to this class of work.

In girders with triangular or lattice-webs, Figs. 33, 35, and 36, the stresses upon the flanges do not increase so gradually as in plate-girders, for the increments of stress accrue only at the junctions of web-bars with the flanges, whereas in the plate-girder they come on at every pair or set of rivets connecting the web angle-irons with the flanges.

In Fig. 33, for instance, the flange stress is constant from 1 to 2, or from A to 7 in the bottom flange, so the area of any member forming the base of a triangle should be constant throughout its length. The stresses may be calculated by the principle of moments, those on the top flange being taken about the points 7, 8, 9, 10, and 11, and those on the bottom flange about the points 1, 2, 3, 4, 5, and 6. The stress on any part of either flange will be found by taking the difference between the moment of reaction of a point of support and the sum of the moments of

the intermediate loads, and dividing it by the depth of the girder.

The upper flange must be treated as a series of struts *if* of considerable length between the junctions with the web-bars, but in those girders in which a close lattice-web *is* used this is not necessary, as the conditions will *closely* approximate those which obtain in a plate-girder.

CHAPTER V.

JOINTS AND CONNECTIONS.

For bridges over certain moderate spans plates cannot be obtained equal to the whole length of the flanges, and therefore these must be made up of several lengths of plates joined together by cover-plates. Wrought-iron bridge plates are not often rolled more than 21 feet long, and some of highest qualities are kept down to 18 feet long. Steel plates may be rolled much longer—up to 40 feet—but these great lengths are awkward to handle, and it is sometimes more economical to use shorter ones, although this involves a greater number of cover-plates. The simplest kind of plate or bar joint is shown in Fig. 51, in elevation. In this *A* and *B* are two plates meeting at *E*, and held together by a cover-plate *C D*, riveted to both of them, as shown by the rivet-heads *f*. A stress acts upon the plate *A* in the direction of the arrow, and puts it in tension; this, unresisted, would draw it away from the plate *B*, and the duty of the cover-plate is to take up the tensile stress and transmit it to the plate *B*. So that there may be no waste of material the joint must be equally strong with the plates it connects. The whole strength of the plates in their solid parts cannot be carried through the joint because there is an unavoidable loss through rivet-holes for the rivets connecting the main plates with the cover-plate.

Let this joint be made between two bars 4 inches wide by

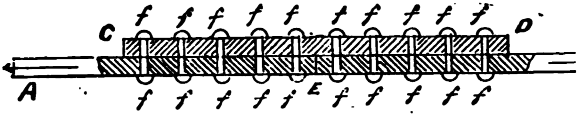


Fig. 51.

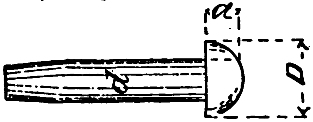


Fig. 52.

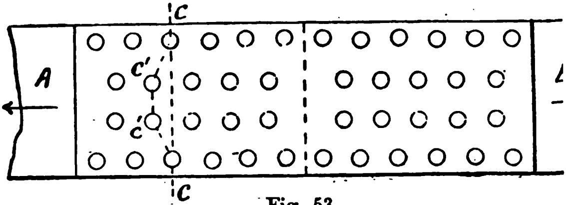


Fig. 53.

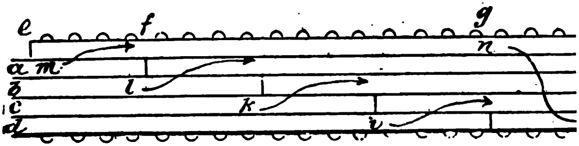


Fig. 54.

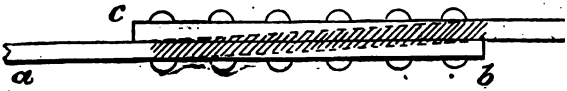


Fig. 55.

$\frac{3}{4}$ inch thick; the stress will tend to pull the bars apart by shearing through the rivets connecting them with the cover-plates, it is therefore obvious the sum of the sectional areas of the rivets on each side of the joint multiplied by the working resistance to shearing, should be equal to the net sectional area of the bar multiplied by its working resistance to tension; the tensile resistance for wrought iron is 5 tons, and its safe resistance to shearing stress 4 tons per sectional square inch.

If the rivets used are $\frac{7}{8}$ inch diameter, the net area of the bar will be $(4 - 0{\cdot}875) \times 0{\cdot}75 = 2{\cdot}344$ square inches, of which the working resistance is $2{\cdot}344 \times 5$ tons $= 11{\cdot}72$ tons, This stress has to be carried by one set of rivets from the bar A to the cover-plate $C\,D$, and thence by another set of rivets from that cover-plate to the bar B, the number of rivets necessary to do this is now to be ascertained.

The cross-sectional area of a $\frac{7}{8}$-inch rivet is 0·6 square inch, therefore the shearing resistance of each cross-section of rivet area will be $0{\cdot}6 \times 4$ tons $= 2{\cdot}4$ tons; hence the number of rivets required on each side of the joint E will be

$$\frac{11{\cdot}72}{2{\cdot}4} = 5 \text{ rivets},$$

for something more than four rivets being required we must use five, as shown in the Figure.

There is a matter connected with riveted joints of this class which I may as well deal with now as later on—it is the value of the friction of the plates to sliding one upon the other. That this friction has some value cannot be denied, but to ascertain its actual value is impossible; under the law of friction it is some fraction of the total force with

which the plates are pressed together by the contraction of the rivets in cooling, and this pressure will depend upon the temperature of the rivets when "closed" or headed up, and on their permanent yield while cooling: on the whole this quantity is so indefinite that it is ignored in designing joints, and shearing area sufficient to take up the whole stress provided.

In order that the work, when finished, shall be sound and solid, it is imperative that the rivets should be properly designed, so that the strength of the heads shall be equal to that of the body.

An ordinary rivet is shown in elevation in Fig. 52; $d =$ diameter in inches; $D =$ diameter of head in inches, and $a =$ height of head in line with the outside of the body—also in inches.

The greatest stress to which the heads will be subject will be that due to the contraction of the bodies in cooling, and this will not exceed 10 tons per cross sectional area of rivet body, for at that stress the limit of elasticity will be reached and the rivet will stretch. In the direction in which a rivet will pull out of its head—parallel to the fibre—the resistance does not probably exceed four-fifths of the shearing resistance across the grain; the working stress along the grain should therefore be taken as $4 \times 0{\cdot}8 = 3{\cdot}2$ tons per sectional square inch.

The maximum pull upon the head of a rivet will, under these conditions, be $0{\cdot}7854\ d^2 \times 10$ tons $= 7{\cdot}854\ d$. The shearing area in the head will be the height a multiplied by the circumference of the body, and its resistance at 3·2 tons will be equal to $3{\cdot}1416\ d \times a \times 3{\cdot}2$ tons $= 9{\cdot}853$ tons. Equating this with the stress we have $9{\cdot}853\ d\,.\ a\,. = 7{\cdot}854\ d^2$, whence $a = 0{\cdot}797\ d$. If pan-head rivets are used this will give the thickness of the heads, but if they

are hemispherical the height at centre will be somewhat more. The head of the rivet must also have sufficient area around the body to prevent the compressive stress from exceeding 4 tons per square inch. The area of this rim of the rivet-head will be equal to $0{\cdot}7854\ (D^2 - d^2)$; and its resistance to compression $0{\cdot}7854\ (D^2 - d^2) \times 4 \text{ tons} = 3{\cdot}1614\ (D^2 - d^2)$. Equating this with the stress upon the rivet-body we have, $3{\cdot}1416\ (D^2 - d^2) = 7{\cdot}854\ d^2$; therefore $D^2 = 2{\cdot}5\ d^2 \times d^2 = 3{\cdot}5\ d^2$, $D = \sqrt{3{\cdot}5\ d^2} = 1{\cdot}87\ d$; so for example the diameter of head of a $\frac{3}{4}$-inch iron rivet should be $1{\cdot}87 \times 0{\cdot}75 = 1{\cdot}4$ inches, that is in commercial size $1\frac{3}{8}$ inches full, so it would be advisable to make them $1\frac{1}{2}$ inches.

The bearing of the rivets upon the surfaces of the rivet-holes has also to be considered, for the compressive force there should not exceed 4 tons per square inch, measured square to the direction of the stress, so that the effective area will be equal to the diameter of the rivet multiplied by the thickness of the plate. The rivets shown in Fig. 51 are in single shear, that is only one sectional area comes into action, had there been another cover-plate below the main plates A and B, each rivet would be in double shear, and therefore only half the number shown would have been required.

If the rivet is in single shear its resistance $= 0{\cdot}7854\ d^2 \times 4 \text{ tons} = 3{\cdot}1416\ d^2$, and if $t =$ the thickness of the plate, the bearing resistance $= 4\ t\,.\,d$; therefore, equating these, we find $3{\cdot}1416\ d^2 = 4\ t\,.\,d$; whence $t = 0{\cdot}7854\ d$, or $d = 1{\cdot}273\ t$.

If the rivet is in double shear the central plate should theoretically be double this thickness, but this is not requisite, because that plate being closely riveted up between two others cannot bulge or thicken without bursting off the rivet-heads, and in actual practice I have never

experienced nor heard of any failure in such a case, although the thickness given for single shearing has not been exceeded when the plates were in double shear. This teaches that if we have sufficient cross sectional area in the rivets to pick up the longitudinal stress on the plates, the bearing in the rivet-holes will do what is required.

In regard to the rivets occurring near the ends of plates that are joined, rupture might occur by tearing off a piece of the end of the plate or by the rivet bursting out of it if too near the edge, and to avoid this latter, experiment and experience show that the distance from the body of the rivet to the edge of the plate in the line of stress should not be less than one and a half diameter of the rivets; thus, with a $\frac{3}{4}$-inch rivet, $\frac{3}{4} \times 1\frac{1}{2} = 1\frac{1}{8}$ inches. The diameter of the rivet is $\frac{3}{8}$ inch, therefore the distance from the centre of the rivet to the edge of the plate should not be less than $1\frac{1}{8} \times \frac{3}{8} = 1\frac{1}{2}$ inches. Generally then the distance of the centre of the rivet should not be less than two diameters from the edge of the plate in the line of stress. Laterally there must be sufficient sectional area of plates as the rivets are themselves proportioned to this.

In compression joints the stress should, if the workmanship were absolutely perfect, pass from the end of one plate to that of the next, upon which it presses, and in such a case the cover-plates would only need to be long enough to keep the main plates opposite each other; but we cannot be certain that these plate ends are in contact uniformly over their ends—the process of riveting may disturb them—and therefore it is usual in joints of the type with which I am now dealing, to put as many rivets in a compression joint cover-plate as would be used if the stress were tensile.

In large triangular-webbed girders, when the top flange

is made in segments of which the ends are faced up truly in a lathe and made to fit accurately there is no need of long cover-plates to the joints of that member, and only enough rivets to keep the contiguous ends in juxtaposition will be required.

If all the rows of rivets in the plates of a girder are kept with the rivets in each row opposite each other, the loss of effective width of the plates will be equal to the number of rows of rivets multiplied by the diameter of the rivet-hole; but some saving may be made by zig-zagging the rivets, as shown in plan in Fig. 53, provided that the pitch of the rivets is sufficiently wide to allow ample room between the rivets of consecutive rows measured direct from centre to centre. The main plates A and B are assumed to be under tensile stress tending to pull them apart. The effective width on the line c c is the whole width less two rivet-holes, but to hold this the rivets must be so arranged that the line c c' c' c, less four rivet-holes, is not less than the effective width across the plate.

If the flange of a girder consists of several tiers of plates, which are long enough to require jointing, a saving may be effected by bringing several joints under one continuous cover-plate, as shown in Fig. 54. The distances between the joints must allow for sufficient rivet area to take up the stress from one plate, and these stresses will be passed on in the following manner: The main plate ends are shown by a b c and d on the one side, and a' b' c' and d' on the other side. The plates are assumed to be all the same thickness, and, therefore, the same number of rivets will be required to pick up the stress from each main plate. The stress on the plate d passes in the direction of the arrow i to the plate c'; that on c in the direction of the arrow k to the plate b'; the stress from plate b along the arrow l to the

plate a'; and the stress from plate a passes along the arrow m through the rivets between e and f to the cover-plate $e\ h$, and thence along the arrow n through the rivets between g and h to the plate d'.

Now if these joints were not brought into this proximity two laps of cover-plate would obviously be required for each joint, and as there are four joints that would necessitate eight laps of cover-plate in all; it is seen from the figure that only five laps of cover are requisite in the arrangement shown.

Fig. 55 shows how the stress upon the plate $a\ b$ is picked up bit by bit and transmitted through the rivets $e\ f\ g\ h\ i\ j$ to the plate $c\ d$. The shading shows the diminution of stress in the lower plate as it increases in the upper. In this and the preceding diagram the thicknesses of the plates are much exaggerated for the sake of clearness, and this may give the impression that, along the arrow n for instance, considerable bending stress would be set up in the rivets, but the actual length of the rivets between heads would only be $2\frac{1}{2}$ inches if the plates were $1\frac{1}{2}$ inch thick.

For these joints to be perfect it is evident that the rivet-holes should be exactly in line and of exactly equal diameters, which condition can be best insured by fastening the plates together at the ends and drilling the holes through all at one operation, then when the rivets are closed up their bodies will be truly cylindrical, and their bearing on the plates uniform throughout.

Hydraulic riveting is universally used for runs of riveting, and great density is secured in this way in the bodies of the rivets, for a steady pressure of some twenty tons will do more to solidify the rivet than any amount of hammering, and the whole pressure being applied the edges of the rivet heads will not be cracked, a not infrequent

occurrence in hand-riveting when the hammering is continued too long.

It may be mentioned in this place that the shearing resistance of mild steel is only about 26 tons per sectional square inch against 21 tons for wrought iron, and this is probably due to the absence of fibre in the steel, for it is only a gain of about 27 per cent., while in tension the steel shows about 30 tons per sectional square inch against 22 tons for ordinary girder iron, or nearly 37 per cent. increase of strength. Steel in this way does not seem to possess the toughness of wrought iron.

Riveting is vastly superior to bolt connections, because it not only ensures a true fit, but will secure permanency of hold, which is not certain in bolted joints; but as bolted connections are constantly required in some part or another of any structure of importance, it is necessary to enquire into the proper proportions to be adopted.

As bolts are put in place cold they are not liable to the exceptional stress put upon rivets during contraction. Let $d =$ diameter of body; $D =$ diameter of head, that is the least width across the head; $H =$ the height of head; and $h =$ the height of nut, the working stresses remain the same as for the iron rivets.

The ratio of shearing area to draw the body out of the head of the bolt to the cross sectional area of the body will be—as there are only the ordinary working stresses to consider—as 5 to 4. Hence $4 \times 3{\cdot}1416\, d \times H = 5 \times 0{\cdot}7854\, d^2$; whence $H = 0{\cdot}312\, d$. It is common practice to make the head of a bolt not less than half the diameter of the body, and the height of the nut equal to the diameter of the bolt to allow for weakening in cutting the thread, which will generally remove about one-third of the stripping surface, which must be sheared off to release the nut. The

effective diameter of a bolt for calculations of strength is the diameter at the bottom of the thread if it is in longitudinal stress, but if it is in shearing stress the cylindrical part of the body gives the effective diameter. The bearing area of the head or nut will be to the cross section as 5 is to 4, $5 \times 0{\cdot}7854\, d^2 = 4 \times 0{\cdot}7854\, (D^2 - d^2)$; therefore $D^2 = 2{\cdot}25\, d^2$, and $D = \sqrt{2.25} \times d = 1{\cdot}139\, d$.

To ensure the fitting of bolts to holes it is necessary that the bolts should be accurately turned, and the holes drilled, or broached out if previously punched, in a drilling or broaching machine, hand-broaching must not be permitted, for it is impossible for it to give true work. If there is longitudinal stress upon the bolts especial care must be used in examining the threads, for upon the solidity of these depends the safety of the work. If the thread is worked up in dies it will be partly cut and partly squeezed up, and not have one-fourth its normal strength, and all bolts and nuts which do not show cleanly-cut threads should be condemned, if required for use under longitudinal stress; though if the stress on the bolt is shearing, a fairly good thread should be insisted upon.

In the matter of screwing up nuts there is a tendency to err in screwing them up too tightly and so putting an unreasonable initial stress upon the bolts, for, with even a short spanner, an enormous multiplication of force in the direction of the axis of the bolt may with slow threaded bolts come into action.

I have known one instance of this which was very striking—in more senses than one—a cast-iron bridge on a northern railway was about to be tested by passing over it a train of very heavy locomotives, weighing with their tenders about eighty tons: the bolts were of large diameter, and a zealous but ignorant inspector, thinking that the

tighter the nuts were screwed up the firmer would be the bridge, took a long spanner, and threw his weight upon its end to bring the nuts "well home" as he termed it; he weighed about sixteen stones; when the load came on the bridge several bolt heads burst off, to the danger of those in the neighbourhood.

All that is necessary in the tightening up of nuts upon bolts is to bring the surfaces of the parts joined into close and firm contact, and if there is any danger of the nut shifting, by vibrations due to heavy loads, the end of the bolt may be lightly riveted over it.

The connections of the different parts of girders is a most important matter, distinct from those connections which are rendered necessary by the impossibility or inconvenience of obtaining plates and bars exceeding certain lengths. After a certain length is reached for any given section of plate, or bar, its cost is increased, and not only this but the cost of labour in handling a heavy piece of metal is greater often, than that which would be incurred in dealing with two pieces of half the weight each; but the matter which now comes under consideration is the connection of the web of a plate-girder with the flanges through the main angle-irons.

Taking any point in either flange, in a girder of moderate size, we find the flange joined to its angle-irons by two rivets in a line transverse to the length of the flange, therefore there are two rivet areas in shear, and corresponding to these the two angle-irons have one rivet connecting them with the web and each other; this rivet is in double shear, and therefore equal to the two rivets which connect the main angle-irons with the flanges.

As the *stress upon* the flanges of a plate-girder increases *gradually from the* points of support to the centre of the

span—under a uniformly distributed load—the rivets pick it up from the web bit by bit, and the angle-irons pass it in the same way to the flanges.

Now it is not sufficient that the total of the sectional areas of the rivets between one point of support and the centre of the span shall be sufficient to transmit the whole stress accruing at that point, because the increase of stress is much more rapid near the points of support than farther on towards the centre as is curve of moments of stress: each pair of rivets joining the main angle-irons to the flanges must pick up the stress accruing between them and the pair of rivets next behind them; the same observation also applies to the single rivet, in double shear, connecting the web with the main angle-irons.

I will take as an example a girder 100 feet span, carrying a uniformly distributed load of 1·5 tons per lineal foot of span. Its depth is assumed to be 8 feet, and the proposed pitch of rivets 4 inches. Now let the first pair of rivets be immediately over the point of support, where horizontal stress ceases, then the next pair of rivets will have to pick up the stress from the first web rivet, which, as the angle-iron rivets are almost invariably zigzagged, will be 2 inches from the point of support.

The horizontal stress at this point will be found by the ordinary formula, making $x = 2$ inches $= 0{\cdot}1\dot{6}$ feet—then

$$\frac{w \,.\, x^2}{2\,d} - \frac{w \,.\, l \,.\, x}{2\,d} = \frac{1{\cdot}5 \times (0{\cdot}1\dot{6})^2}{2 \times 8} - \frac{1{\cdot}5 \times 100 \times 0{\cdot}1\dot{6}}{2 \times 8}$$

$$= 1{\cdot}547 \text{ tons.}$$

At a safe shearing stress of 4 tons per sectional square inch the resistance of the two sections of shear in a $\frac{3}{4}$-inch

rivet will be 0·44 square inches × 2 shearing sections × 4 tons = 3·52 tons, so there is ample strength here.

I will now take the stress at the next rivet, and find the increment to be taken up by it; the value of x will be 6 inches = 0·5 foot, and

$$\frac{wx^2}{2d} - \frac{w.l.x}{2d} = \frac{1\cdot5 \times (0\cdot5)^2}{2 \times 8} - \frac{1\cdot5 \times 100 \times 0\cdot5}{2 \times 8} = 4\cdot664 \text{ tons.}$$

The increment to be picked up by this rivet is 4·664 − 1·547 = 3·117 tons, which is well within the 3·52 tons resistance of the rivet to shearing.

I will take one point at the next rivet to show that the increment of stress will not increase as we progress along the girder; x will now be 10 inches = $0\cdot83\dot{3}$ foot, and

$$\frac{wx^2}{2d} - \frac{w.l.x}{2d} = \frac{1\cdot5 \times (0\cdot83\dot{3})^2}{2 \times 8} - \frac{1\cdot5 \times 100 \times 0\cdot83\dot{3}}{2 \times 8}$$
$$= 7\cdot748 \text{ tons.}$$

So the increment to be picked up by the rivet will be 7·748 − 4·664 = 3·084 tons, which is smaller than that which accrued to the previous rivet.

There is another source of stress upon the rivets holding the flange-plates together which must be considered, when the flanges consist of two or more tiers of plates. As the plates farthest from the neutral axis of the girder carry the heaviest stress per sectional square inch, it is obvious that, by their elastic resistances, they will tend to slide upon those next beneath them, and in so doing will bring shearing stress upon the rivets holding these plates together, the intensity of the stress being equal to the difference of stress upon two contiguous plates. To take an extreme case: let the girder be 12 inches deep over all, and let each flange

consist of two plates 12 inches wide and ¾ inch thick each, then the centres of the thicknesses of these plates will be respectively 5·625 inches and 4·875 inches from the neutral axis; so if the outer plate is under a stress of 5 tons per sectional square inch, the total stress upon it will be—deducting the loss by rivet-holes—$(12 - 1{\cdot}5) \times 0{\cdot}75 \times 5 = 39{\cdot}375$ tons; and the stress on the inner plate will be $= 39{\cdot}375 \times \frac{4{\cdot}875}{5{\cdot}625} = 34{\cdot}125$ tons. The difference is $39{\cdot}375 - 34{\cdot}125 = 5{\cdot}25$ tons to be carried by each pair of rivets. To carry this two ¾-inch rivets would be insufficient, but two extra rows outside the angle-irons and zigzagged with the inner rows may be added; the resistance of the four rivets would be 7·04 tons.

Where joints occur in the web-plates the shearing stresses at those points must be taken up by cover-plates, or by tee-iron stiffeners, which act also as covers to the web-plates; if these will not accommodate a sufficient number of rivets, cover-plates of greater width must be used between them and the web-plates. The shearing stress at any section of the web is equal to the load per lineal foot of span multiplied by the distance of that section in feet from the centre of the span. The web-plates will have their lengths vertical in a girder as deep as 8 feet, and would be about 4 feet in width, so if there is a 2-feet bearing on the supports at each end of the span, the first web joint will be 2 feet from the edge of the support, and therefore 48 feet from the centre of the span. The shearing stress on this section will therefore be equal to 48 feet × 1·5 tons = 72 tons, which at 4 tons per sectional square inch will require a total rivet area of $72 \div 4 = 18$ square inches. If the rivets are ¾ inch in diameter the sectional area is 0·44 square inches and therefore the number of rivet sections necessary will be

$18 \div 0{\cdot}44 = 41$, on each side of the joint. As each rivet is in double shear, 21 rivets will be required on each side of the web joint. The height—or length—of cover-plate can only be taken between the main angle-irons, which would not be less than 4 inches $\times$ 4 inches for a girder of this span, thus cutting down the length of web cover to 8 feet, less 8 inches $=$ 7 feet 4 inches $=$ 88 inches. If the rivets are set out for 4-inch pitch the number will be $88 \div 4 = 22$ rivets, which will give just what is wanted. As the centre of the span is approached the shearing stress falls off, but the number of rivets would not be reduced, for to ensure solid work with light plate-girders, the rivet pitch should not exceed 4 inches when $\frac{3}{4}$-inch rivets are used; with heavier plates and angle-irons using $\frac{7}{8}$-inch or 1-inch rivets, the pitches may conveniently be increased to 5 and 6 inches.

CHAPTER VI.

DEFLECTION.

THE deflections of girders can, as a rule, be only approximately calculated, as the modulus of elasticity of the material to be used is not specified, and also the manufacture will give wide differences; however, assuming the workmanship to be good, formulæ may be found for the probable deflection.

Fig. 56 shows a diagram of a deflected girder AB, of which $a\ b\ c$ is the flange, and $g\ f\ h$ the bottom flange. These flanges are taken as designed to suit the variations of stress from a uniformly distributed load, and therefore under a constant stress per sectional square inch throughout their lengths. Under these conditions the curve of deflection will be circular. Let e be the centre of the arc of deflection, and from this point draw the radii $e\,g$, $e\,h$, to the points where the girder rests upon its supports, which are assumed to be at the same level. Join $a\,c$, and from the centre e let fall the perpendicular line ef, cutting $a\,c$ in the point d.

Let $l = a\,b\,c =$ the length of the girder; $d = bf =$ depth of the girder; $R = e\,a =$ radius of curvature; $D =$ deflection at centre $= i\,b$; $L =$ difference in length of flanges after deflection $=$ sum of the compression of the top flange [illegible]xtension of the bottom flange.

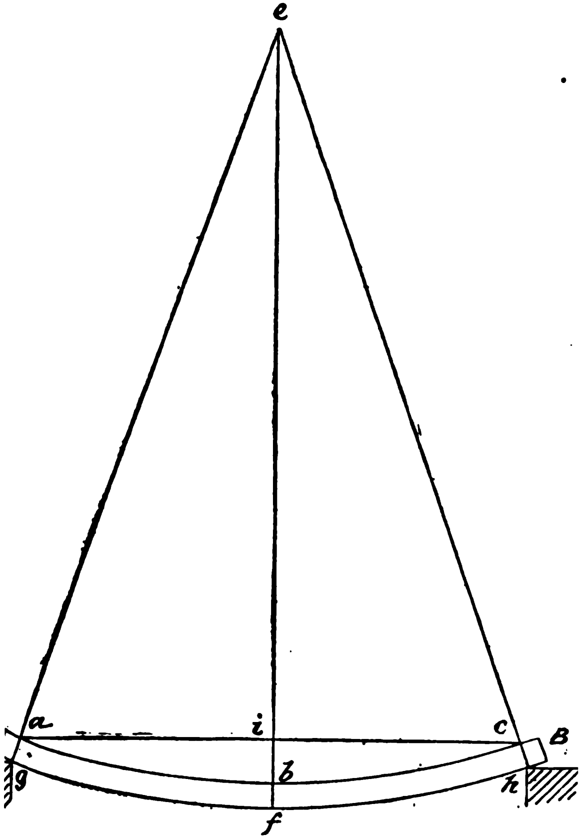


Fig. 56.

The deflection being very small in comparison with the radius of curvature, it may be assumed that $i\,e = a\,e = R$; and $a\,c = a\,b\,c = l$. Then (Euclid, prop. 35, Book III.), because if any two right lines contained in a circle intersect one another, the rectangles formed by the segments of such lines are equal; and the diameter $2\,R$ is intersected at the point i by the chord $a\,e$; therefore,

$$2\,R \times D = a\,d \times d\,c = \frac{l^2}{4}$$

because $a\,d = d\,c = \frac{l}{2}$, and $\left(\frac{l}{2}\right)^2 = \frac{l^2}{4}$; whence $D = \frac{l^2}{8\,R}$

Then by the properties of similar triangles;

$$R : d :: l : L,$$

which we also know to be true, because the circumferences of circles are in direct ratio to their radii; wherefore

$$(R \times d) : g\,f\,h :: R : a\,b\,c,$$

but $g\,f\,h = a\,b\,c \times L$; and $R \times d : a\,b\,c + L :: R\,a\,b\,c$.

Whence, as $a\,b\,c$ is taken equal to l, we find in either case $R\,L = d\,l$, therefore

$$R = \frac{d\,l}{L}. \quad \text{But } D = \frac{l^2}{8\,R}; \text{ therefore } R = \frac{l^2}{8\,D} = \frac{d\,l}{L}; \text{ and}$$

$$D = \frac{L\,.\,l}{8\,d};$$

which is the formula for the deflection of the girder at the centre of the span.

In a flanged girder uniformly loaded, and with uniform stress per square inch, the same on both flanges, let $s =$ the stress per sectional square inch; $a =$ the area of one flange

in square inches; W = the uniformly distributed load; and E = the modulus of elasticity in tons.

The shortening of one flange and extension of the other will be equal to the length of flange—which is in stress all along the span = l—multiplied by S and divided by E, so, adding the compression and extension together, we find,

$$L = \frac{2.s.l}{E}$$

therefore,

$$D = \frac{L.l}{8d} = \frac{2.s.l}{E} \times \frac{l}{8d} = \frac{s.l^2}{4dE}$$

But the stress on either flange at the centre of the span, under a uniformly distributed load is

$$= \frac{W.l}{8\,d} = s \times a\text{; therefore } s = \frac{Wl}{8.d.a}$$

whence,

$$D = \frac{s.l^2}{4d.E} = \frac{W.l}{8da} \times \frac{l^2}{4dE} = \frac{Wl^3}{32d^2.E.a}$$

The modulus of elasticity determined from the effect of transverse stress upon different sections of metal is shown in tons per sectional square inch in the following table—

MODULUS OF ELASTICITY.

Cast iron rectangular bars—1 brand .	E = 6,785 tons.
,, ,, ,, ,, mixed .	,, = 8,434 ,,
,, ,, round and square tubes .	,, = 5,453 ,,
,, ,, I girders	,, = 5,893 ,,
Wrought-iron rolled floor beams . .	,, = 7,304 to 9,630 tons.
Single webbed plate-girders (iron) .	,, = 6,391 tons.
Tubular plate-girders (iron) . .	,, = 10,541 ,,
Conway tubular bridge (iron) . .	,, = 8,372 ,,
Mild steel (32 tons per square inch tensile strength)	,, = 12,000 ,,

It is almost startling to notice the variation of these factors with different forms, and those for rolled-iron floor beams are only inserted to illustrate this, as they are useless for practical purposes. The 6,391 is the minimum of a large number of railway bridge plate-girders calculated by myself, and for that class of work I have been accustomed to use 6,400 tons in estimating deflections.

For solid wrought iron of the quality used in the girders referred to 11,125 tons is given as the value of E, so if we assume that in steel bridges this factor is deteriorated by being riveted up in the same ratio as wrought iron work; then the modulus of elasticity for built up steel plate-girders will be—

$$E = 12{,}000 \times \frac{6400}{11125} = 6{,}903 \text{ tons.}$$

I will take as an example, a plate-girder 80 feet span; 7 feet 6 inches deep; the sectional area of one flange $= a$ square inches at the centre $= 60$ square inches; and the uniformly distributed load 2·25 tons per lineal foot of span. The total load $W = 2{\cdot}25 \times 80 = 160$ tons; then—

$$D = \frac{W \,.\, l^3}{32\, d^2 \,.\, E \,.\, a} = \frac{160 \times (80)^3}{32 \times (7{\cdot}5)^2 \times 6400 \times 60}$$

$$= 0{\cdot}118 \text{ foot} = 1{\cdot}416 \text{ inches.}$$

In order then, that under a maximum load the girder shall not deflect below the horizontal line joining its ends, it should be built with a camber of $1\frac{1}{2}$ inches at the centre —common practice allows 1 inch of camber for every 40 feet of span.

We must now consider the deflection of girders which have a uniform sectional area throughout the length, then the stress per sectional square inch will vary, and the

rate of shortening and extension of the material will be different at different points in the span.

Let $I =$ the moment of inertia of the section of the girder, then the moment of resistance is—

$$M = \frac{2\,s\,.\,I}{d} = s \times a \times d. \quad \text{Therefore } a = \frac{2\,.\,I}{d^2}$$

replacing a by this value in the equation for D, we get,

$$D = \frac{W\,l^3}{32\,d^2\,E} \times \frac{d^2}{2\,I} = \frac{W\,l^3}{64\,E\,1}$$

When, however, the section is constant throughout the length, the deflection will not be in a circle, but in a parabolic arc, and therefore greater in proportion to the length. As the stress on the girder varies as the ordinates of a parabola, the mean stress will be two-thirds of the stress at the centre, and the difference in length of the upper and lower surfaces will also be less in the same proportion, and s being the stress at the centre

$$L = \frac{2}{3} \times \frac{2\,s\,.\,l}{E} = \frac{4\,s\,.\,l}{3\,E}$$

then,

$$D = \frac{L\,.\,l}{8\,d} = \frac{4\,s\,.\,l}{3\,E} \times \frac{l}{8\,d} = \frac{s\,.\,l}{6\,d\,.\,E}$$

and,

$$D = \frac{W\,.\,l}{8\,d\,a} \times \frac{4\,l^2}{24\,d\,E} = \frac{W\,l^3}{48\,d^2\,E\,a}$$

Replacing a by its value — $\frac{2\,I}{d^2}$ we get

$$D = \frac{W\,l^3}{48\,d^2\,E} \times \frac{d^2}{2\,I} = \frac{W\,l^3}{96\,E\,1},$$

which must be further corrected for the different form of

the curve of deflection as there is not a constant radius. The equations to the lengths of the different curves in question can be found in any text book on conic sections.

The corrected expressions for different distributions of loads, when the section of the beam is the same throughout its length, are here summarized—

For a cantilever fixed at one end and loaded at the other,

$$D = \frac{W . l^3}{3 E . 1.}$$

For a cantilever fixed at one end and uniformly loaded,

$$D = \frac{W l^3}{8 E . 1.}$$

For a beam supported at both ends and loaded in the middle,

$$D = \frac{W l^3}{48 E 1}$$

For a beam supported at both ends and uniformly loaded,

$$D = \frac{W l^3}{76.8 E . 1.}$$

For girders with triangular webs, a different course will be followed. In this form the flange sections can be more closely adjusted to the stresses to which they are subject, than is practicable with the flanges of plate-girders, because the stress is constant for the length of each bay, and therefore the exact compression, or extension, can be determined for each length, and these extensions and compressions being added together, the difference of lengths of the flanges is found.

The centre lines of the flanges in each bay will form a series of chords to the curve of deflection, so their inter-

sections will be points in the curve of deflection, which, in this case, will be a circular arc; therefore the deflection at the centre of the span will be found from the expression,

$$D = \frac{L \,.\, l.}{8 \,.\, d}$$

The formulæ for deflection are in some cases of great use in determining relative deflections in girders which are combined in such a way that one or two shall assist a series of others by distributing a load which would come upon one only of the series, if no auxiliary girders were used. If all the girders in the combination are made from one and the same parcel of iron or steel, it is to be assumed that all the girders, if the workmanship is of the same quality and description throughout, will have the same modulus of elasticity; it is then an easy matter to so proportion the sections that the stress per sectional square inch shall be practically uniform throughout the work.

In measuring the deflections of bridges and bridge girders, very great care is necessary in order to obtain accurate results; it may seem a very simple matter at first sight, but it is not so in reality.

There are two methods commonly used for reading the deflections; one by a level and staff as in ordinary levelling; the other by having two rods socketed together so that one can slide upon the other, these being placed vertically under the girder to be tested, and slid apart until the upper end touches the under side of the girder, while the lower one rests on the ground beneath; a fine pencil line is drawn on the lower rod and against the bottom of the upper one, the test load is then run on to the bridge, this depresses the upper rod, and another fine pencil line shows the altered position of the upper rod—the distance between the two

pencil lines is the deflection of the girder. Now, after the rods are set, those holding them must not alter their positions about, or they may cause a depression or a rise of ground, which would vitiate the reading, and if the ground is soft a large stone should be placed to receive the end of the lower rod.

Whichever method is used it must be remembered that the abutments themselves yield under a load, so the subsidence at the abutments must be taken simultaneously with that at the centre of the span, and deducted from it to give the deflection of the girder. The deflection of the cross girders can be found at the same time by having the rods under them, and deducting the drop of the main girder from that shown by these rods.

In testing girders prior to erection, the deflections are taken for stationary loads only, and then the deflections at regular intervals along the span are measured, and the curve of deflection plotted to show whether it is uniform, for any jerk, so to speak, indicates a local defect in the workmanship.

CHAPTER VII.

THE PREPARATION OF DRAWINGS.

To ensure the successful and economical execution of any bridge—or for that matter any other work—it is absolutely necessary that the engineer to whom such work is entrusted should be thoroughly conversant with theoretical calculations, with details of construction, and with processes of manufacture.

The engineer referred to will not necessarily be the chief engineer of, say, a railway, one whose time is occupied by general direction and supervision of all the departments within his scope; but rather the head of the drawing-office attached to the bridge department. The chief draughtsman in a bridge and girder yard requires all the same qualifications, as very often the work of nominal engineers is designed by the staff of a contractor.

Now, as I have said, these two men, with the same knowledge and business aptitude, occupy positions widely different, and it may be no loss of time to point out in what way their duties diverge, for it often happens that one man at different times alternates in the two positions; this year he may be assistant manager and chief draughtsman in an iron works, and five years hence he may be senior assistant to a civil engineer, or may be acting as designer and prac-

tical constructor to a number of civil engineers, whose duties leave them no time for personal supervision of designs and detailed drawings. The engineer who is acting for the purchasers will seek in his designs to secure the best work at a fair cost, he will consider labour as well as material, and will study to reduce the weight of the latter so long as the saving thus effected will not be outweighed by extra cost of labour; and he will so arrange that the strengths he specifies for his materials are obtained.

The engineer to a manufacturer must look to the kind of work which pays best, when he has a chance of submitting designs to the purchaser, and this often occurs in competitions; a civil engineer with a good practice would consider it *infra dig.* to enter in competition; but with contractors it is mere business, and—unless the competition is a mere cloak to cover a piece of pre-arranged jobbery—the cleverest or most experienced man will win it.

From the manufacturers' point of view much must depend upon the plant at his disposal, the extent and variety of which will determine the relative cost of labour; so that where in one yard it would pay to use the best material and save weight by the use of adequate machinery, in another it may be necessary to use more material because the plant is deficient, and hand labour necessary for many operations which in the better-equipped works are more cheaply executed by machinery.

In dealing with drawing-office work I shall indicate the course to be pursued to ensure satisfactory results, and not only that, but steady and undisturbed progress in the workshops.

It is assumed that the engineer has, or takes care to obtain, full particulars of the work to be designed: having *these*, he will first make his calculations, commencing where

the work will finish; that is, when the permanent work is complete, and is supporting a maximum load.

The moving or useful load is the first taken in this order, then comes the weight of the road material and floor covering, and these together give the load upon the floor girders of the bridge; these, having been calculated, their weight is added, and thus the load found upon the main girders.

In dealing with this part of our work, I think it advisable to mention some methods of office routine, which I have found not only convenient at the time, but serviceable afterwards; and one is the keeping of a calculation book for each large work dealt with.

If any question of strength should at some subsequent time be raised, a reference to such a calculation book will show what data were supplied, from which to design the bridge, and if there is any error it will readily be seen how it has arisen; if, however, in the intervening time there has been a change of conditions, this is another matter altogether, and one of great importance, as it shows that a bridge which has a good margin of safety now, may not be in the same position five years hence.

This remark applies to both railway and road bridges, for the tendency has been, and is, to increase in the weight of railway rolling stock, and roadway bridges may not only have to carry increasing loads in traction engines, road-rollers, and steam ploughs, but also in the near future heavy motor cars, which, whether they are driven by oil or steam-engines, or electric motors driven from storage cells, must be much heavier than the ordinary vehicular traffic of to-day, and in many cases the loads will be more concentrated.

I mention these matters to show the necessity of close inquiry into the possible loads that may come upon a bridge,

so that nothing may be overlooked in calculating the sections of its component parts.

All the preliminary questions having been satisfactorily answered, the engineer can sit down and design his work in comfort, free from the worry of doubt. The method of working out the calculations will necessarily vary according to the proclivities of different designers; some will keep closely to simple algebraical calculations, while others may use the graphic methods wherever they are applicable. Any stress which can be determined by the parallelogram of forces may also be found by the principle of "moments," and every student should make himself familiar with both processes, for then, in after life, if he has no reliable coadjutor to check his calculations, he can assure himself of his own accuracy by applying first one method and then the other, independently to each case; if the results agree he will be certain that his work is accurate. It is very possible for any man to make the same mistake twice in working out one and the same set of figures, unless they can be displaced, as in the multiplication of several factors, and this has led to the notion which many people hold, that a man cannot check his own work.

If a check by an assistant is to be relied upon, he must be directed to work out the calculations independently, and not have the figures of the first calculator before him. I do not for a moment impute dishonesty in this matter, but some are apt to pass the figures before them as accurate, without making the calculations completely over again. If the stresses upon the component parts of a triangular-webbed girder are first determined by the parallelogram of forces, and then checked by the principle of moments, perfect confidence can be placed in the results, assuming, of course, that they will agree. In the case of a plate-girder the stress at

the centre of the span can be checked by working it out in figures by the two formulæ

$$\frac{w x^2}{2 d} - \frac{w l x}{2 d}\text{; and } \frac{w \cdot l^2}{8 d}$$

After the central stress is determined the cutting down of the plates may be calculated and a side elevation of the flange drawn from the formulæ already given, and the accuracy of this, tested by describing a parabola passing through two points on the inside of the inner tier of plates, one of each immediately over the point of support, and through a point in the outer surface of the outside plate at the centre of the span: if this parabolic arc is enclosed by the profile of the plates, their distribution is correct.

These methods of checking one's own work are very valuable, because they relieve the responsible designer from the anxiety which preys upon him when his assistants do not take any interest in the work, for, as in such cases he cannot rest satisfied until he has re-checked the calculations, he will lose more time than if he dismissed his assistant and relied solely upon himself.

After having entered in the "calculation book" the data supplied, there will follow the calculations for each girder, with a sketch of its cross-section and of its mode of connection with the girder upon which it rests, or to which it is suspended, with the proper number of rivets shown in each joint, and where joint-plates are used, as is often done in triangular-webbed girders, the shapes and sizes of these should be also entered in the calculation book.

The next step is to make the working drawings, and in the preparation of these every care is to be taken that nothing is left obscure, put every dimension and rivet spacing upon the details, and leave nothing to be scaled, and

let the dimensions put on the drawings be calculated, not scaled in the office, except when full-sized details are supplied; which, however, should not be necessary if the drawings are made to a scale sufficiently large to allow room for all the dimensions to be legibly written on them.

An absolute agreement between the drawings and the specification must also be ensured, for any difference, real or apparent, between them will lead at least to considerable delay, for the contractor will not proceed until he has learnt from the engineer which is right, and matters of this sort are not always easily settled by correspondence. As a general rule it is best to confine the specification to qualities of materials and workmanship, and let all the dimensions be on the drawings only, except the title dimensions of span, width, and angle of obliquity if the bridge is not square to the road, river, or railway over which it is to be built; then there can be no clashing in this respect.

The scales to which the contract drawings are to be made will necessarily vary with the size of the structure to be erected; for elevations and plans of bridges up to about 200 feet span $\frac{1}{4}$ inch to a foot is a fair size; the cross section of the bridge should be $\frac{3}{4}$ inch to the foot, and the details $1\frac{1}{2}$ inch to the foot. Sections should be shown of all structural parts, and especially where joints occur, and where there is a connection of two members at an angle to each other—as, for instance, a cross girder with a main girder—the mode of making the joint may be more clearly shown in isometrical perspective, than by the flat plan, section, and elevation. The distribution of the flange-plates may be clearly shown on a general elevation of a girder, by drawing them to a distorted scale, that is, while keeping the lengths to the normal scale of the drawing, making the thicknesses sufficient to allow room for the size

of each plate to be written on. This is shown in Fig. 57; *a b* shows the inner tiers of plates in the top and bottom flanges; *c d* the next tier—in this case there are only two tiers of plates—and *e f* is the cover-plate to the joints *k k*. The flanges are shown removed a little from the plate-web *g k*, which, with the main angle-irons and the stiffeners *i i*, &c., is drawn to a normal scale.

In all the drawings, and especially in the details, accuracy of form should be studied and adhered to. The drawings given to the workmen are more easily read by them if the sections are shown as they really are, than if they are slovenly in outline or inaccurate in profile. In Fig. 58 are shown some common sections in constant use in bridge work. The full lines show the actual sections as they are rolled, and the dotted lines show them as they are sometimes drawn by inexperienced draughtsmen. *A* is the cross section of an unequal-sided angle-bar; *B* that of a tee-bar; *C* that of a channel-bar; and *D* that of a rolled girder. All these have a taper or "draught" to enable them to leave the rolls easily, as well as to ensure a uniform solidity throughout the section. Further, it may be observed that if the limbs of these sections were rolled parallel with sharp re-entering angles, the "roots" at *a* would be weakened, a most serious matter in angle-irons which connect together plates at right angles to each other, and equally so with channel-bars when used as webs and angle-bars combined in compound girders. The rolled girder *D* could not be expected to leave the rolls without the "draught" shown on its flanges.

In scheduling such bars for ordering from the mills—and this is a drawing-office duty—a consistent system must be followed. Tee-bars are specified with the table dimension first. Thus, say, No. 18 tee-bars 6″ × 3″ × 4′ 0″ long; if

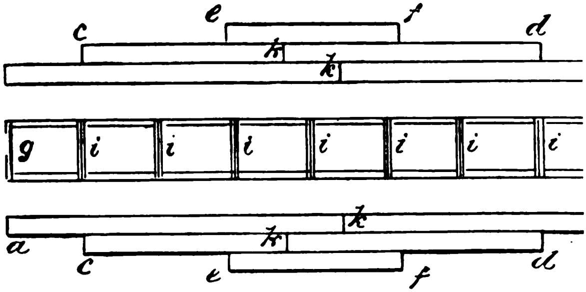


Fig. 57.

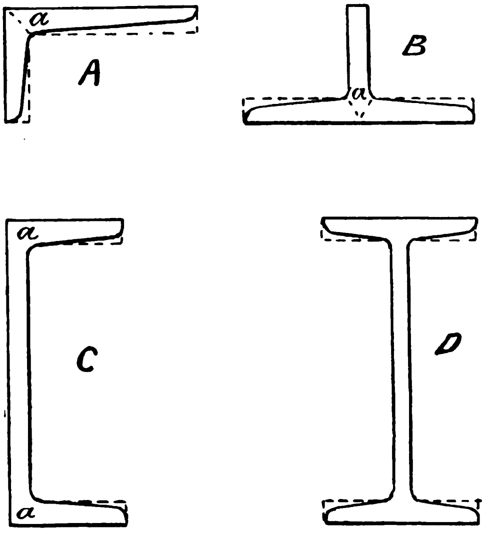


Fig. 58.

this were written 3″ × 6″ for the section, it would mean that the table is 3″ wide and the web 6″ high, and, as both forms are in constant use, an error might easily be made here. The safest way is to put sketches with figured dimensions on the margins of the schedules. It does not matter in ordinary angle-bars from the rolls which dimension comes first, but—I suppose from association of ideas—I always put the size of the horizontal limb first, that in the figure would be 5″ × 3″; but if it were intended to be used with the short limb horizontal it would appear in the list as 3″ × 5″. When it is considered that these lists are used for reference in the drawing-office, the convenience of such a practice becomes self-evident. I have omitted the thickness in the above examples for a reason I will now show. The bars can be ordered by thickness, or by the weight per foot; when thickness is specified, that will be mean thickness. Thus an angle-steel may be specified as 5″ × 3″ × ½″ thick; or as 5″ × 3″ by 12·75 lbs. per foot run; a tee-steel as 6″ × 3″ × ½″ thick, or as 6″ × 3″ × 14·45 lbs. per foot run, and it is safest to use the weight in specifying, and to mark it on the drawings. We know the sectional area required, and from this can calculate the weight per lineal foot. The rolls are adjustable, so there is no difficulty in getting very nearly the weight specified. Some rolling margin must be allowed to the manufacturers, but—except in the rolled-joist trade—this need not be large. The channel-bar *C* might, if steel, be specified as 8″ × 3″ × 3″ × ½″ thick, or 8″ × 3″ × 3″ by 22 lbs. per foot.

In rolled-joist lists the heights are specified first and the section fixed by the weight per lineal foot; thus the joist *D* might be 8″ × 4″ × 19 lbs. per foot, or 8″ × 4″ × 25 lbs. per foot, as both sections are in the stock lists. These

joists are principally used for builders' work, but may occasionally be found in bridge work; as there is not much to be said about them, what there is may be said here. We have two markets to choose from, the English and the foreign, the latter being Belgian and German. Where rolled joists are used it is cheaper to use English, as the strength is greater in proportion to weight than is the English price to the foreign. A comparison between two rolled joists, one English and the other foreign, will show this. In choosing sections we are tied to those rolled by the makers, so it is useless to discuss whether they are well or ill-designed; they must be considered as they are. For convenience of calculation the spans will be taken as ten feet, and the strengths taken from the published lists of the merchants and manufacturers. Now, if two rolled joists are made from the same charge of metal, and their proportions are exactly similar, their strengths will be in exact proportion to the weights per foot run, so that the quotients found by dividing the loads by the weights of the girders would be equal; from this it follows that—the same factor of safety being taken in both cases—by dividing the safe loads on joists by their weights per foot run, we find a series of critical numbers which give the relative values of the joists, they show the comparative efficiencies per pound of weight per foot of length. Taking a 12-inch by 6-inch section of each make, and 3 as the factor of safety, the English has a weight of 54 lbs. per foot, with a carrying strength of 43·84 tons; the critical number is $43{\cdot}84 \div 54 = 0{\cdot}811$. The foreign section has a weight of 45 lbs. per foot, with a carrying strength of 22 tons; the critical number will therefore be for this one $22 \div 45 = 0{\cdot}488$. In both cases the material is steel. In *this case, if* the foreign material is worth £5 per ton, the

English is worth £8 6s. Other examples show similar results. As there is nothing approaching this difference in price on the actual market, it follows that although foreign steel can be delivered in our own manufacturing centres at a lower price than that at which we can produce it on the spot, yet the quality of the English steel joists is so much superior, that it is more economical to use them than the cheaper ones in price per ton. It is only fair to mention that in the very light joists the advantage is not quite so great; with a section of $4\frac{3}{4}$ inches by $1\frac{3}{4}$ inches the relative values are £5 and £7 5s. per ton.

It will have been gathered by my readers from the preceding matter that when I speak of a draughtsman, I mean a man who understands his work and can design it without instruction, and not a person who has only learnt enough geometrical drawing to put lines on paper when he is told where to put them. The term engineer-draughtsman is useless. A man is an engineer or he is not, and "engineer" covers the other term in a bridge-designer's office.

In regard to the calculations required in the drawing-office, the course pursued will depend entirely upon the education of the draughtsman and his own studies. If he has mastered the higher stages of mathematics, his work will be so much the easier and more interesting; whether this class is increasing really it is difficult to say, so many students merely "cram" for examinations, and lose the hastily-snatched accomplishments in a year or two. I think there can be no doubt that the most practically solid mathematician for engineering work is he who, while following the constructive processes in the shops, occupies his leisure with the study of mathematics in their application to his work, and also the physical sciences which are allied to engineering.

My endeavour has always been to reduce the calculations to the simplest form consistent with the attainment of logical conclusions; thus, in treating of the theory of the triangular and lattice-webbed girders, I use the relations of the actual lengths of the bars to each other in place of the trigonometrical equivalents affected by more academical writers, and avoid the introduction of the infinitesimal calculus by geometrical investigations which reach the same end, and are much easier to understand. I wish particularly to help those who are pursuing their own studies unaided.

In the calculation of dimensions the most frequently used formulæ are those applying to right-angled triangles (Euclid, book i., prop. 47), in which the sum of the squares on the sides containing the right angle is equal to that of hypothenuse which is opposite to it. For example, let the length of a bracing bar which joins the opposite angles of a rectangular bay in a bridge floor be required. All dimensions are taken to centre lines in planning, and any shortening of bars from their intersections are calculated afterwards. Let the width of the bridge from centre to centre of main girders be 17 feet, and the width of bay 12 feet, measured parallel to the main girders; then the right angle is contained between the width of the bridge and the length of a bay, and therefore the length of the diagonal will be thus found, calling it $= L$

$$L^2 = (17)^2 + (12)^2, \therefore L = \sqrt{(17)^2 + (12)^2} = \sqrt{433}$$
$$= 20{\cdot}808 \text{ feet} = 20 \text{ feet } 9\tfrac{7}{16} \text{ inches.}$$

Here the point arises, how closely can we approximate to true dimensions in actual practice? The answer is simple, it depends upon the price to be paid for the work. We certainly ought not to be more than $\frac{1}{16}$ of an inch out any-

where in fitted work, but here the personality of the workman is a factor; a man who is in the habit of working accurately, will always so set out his work without difficulty, while one of a slovenly nature will be out by all sorts of differences. I have known one manufacturer who had his drawings marked for rivet spacing in joint plates to $\frac{1}{64}$ of an inch, but he did not get it in execution. When we come upon a dimension which does not admit of being stated in the figures ordinarily used in the shop, if it is alone, we must take the nearest to it, or supply a full-sized detail to show it, but if it refers to a line of rivets, a number of pitches can be taken that will make an even number of eighths or sixteenths, that dimension marked, and a note put on the drawing, "to be divided into number . . . pitches."

An instance of this sort occurred in connection with a large railway bridge over the Thames in London. In order to get the necessary camber, the top flange had to be made longer than the bottom one, and it was determined to do this by making a difference in the rivet pitches throughout the length of each girder, having, of course, equal numbers of rivets in the top and bottom flanges, in one row. The rivets in the top flange are 4 inches pitch, and those in the bottom flange 3·995 inches pitch. In a 20-foot run on the top flange, there would be 60 rivets, and in the bottom flange the same number would occur in a length of 19 feet $11\frac{11}{16}$ inches and $\frac{1}{80}$ of an inch.

Before commencing the calculations of strength the factors of safety must be determined, and the quality of material to be used. The Board of Trade regulations fixing a definite safe stress for iron, and another for steel, both in tension and compression, and without regard to the quality of the material used, hardly suits the scientific mind, even

when it is counter-checked by a limit to deflection in relation to span.

This question of factors of safety requires very careful consideration in connection with the quality of material to be specified.

Adopting the same working stress in both tension and compression, is justified on the assumption, that the modulus of elasticity is the same whether the metal is extended or compressed, and this would indicate that the limit of elasticity, rather than the ultimate strength, should guide us in fixing the working stress. We have, however, dropped into the habit of applying the factor of safety to ultimate strength, and so it must remain at present.

The making an allowance for vibration is a mere farce, as there are no data upon which to base any figures, and the only way in which we can deal with vibratory deflection is to determine whether the extreme deflection corresponds to a load which would endanger the structure; and this deflection can only be ascertained after the bridge is completed.

What we have to do is to ascertain the quality of material usually obtainable in the neighbourhood of the works to be designed, and specify nothing outside of this. In Staffordshire, we may safely specify 22 tons per sectional square inch for bridge or girder iron, and farther north, in the Cleveland districts, 21 tons for tension, and as factor of safety, I should use 5; giving for the former 4·4 tons as the working stress in tension, and for the latter 4·2 tons per sectional square inch. Steel is obtainable at strengths having a wide range, but for bridge work it should be specified not less than 28, or more than 32 tons per sectional square inch in tension. In specifying iron, there is no need to fix an upper limit, as an excess of tensile strength in

that material does not imply increased brittleness; but the mild steel, being of a composite nature, necessitates a more exact description. The lower limit, 28 tons per sectional square inch, would be taken in the calculations, and the working stress would therefore be $28 \div 5 = 5{\cdot}6$ tons per sectional square inch.

In regard to compression, the whole matter turns upon the crippling limit of the material, not upon its ultimate resistance to absolute crushing, and we are therefore obliged to rely upon formulæ of a more or less empirical character; those that I give have stood the test of more than 35 years practice under my own observation.

All the preliminaries being settled, we can now turn to the making of the drawings—a simple matter, but one on which a few hints may be useful.

You can never be sure that the drawing boards provided in an office to which you are newly introduced are true, therefore, unless you have leisure to test this, do not use the tee-square on two sides of the board, and as a general rule it is better to keep the tee-square for the horizontal lines, and use set squares worked upon it for the vertical lines; getting out of square is an infinite source of discomfort to the draughtsman, for then his drawing will not agree with the calculated dimensions marked upon it.

In a large and well-appointed drawing office, sheets of drawing paper will be kept mounted upon the boards ready for use, covered over with thin paper until they are wanted; but in smaller establishments the drawing paper will usually be taken as required and pinned to a board, no one having leisure to damp and mount it.

The draughtsman has to work on what quality of paper is supplied to him, and must suit his pencils to its texture;

for choice, he would have Whatman's paper mounted upon brown holland, which is certainly the best for contract drawings: but if this is not available the best must be made of what is at hand.

In setting out the centre lines of structures any constructional lines necessary to plot them should not be put upon the paper on which the finished drawing is to be made. This work should be done upon a sheet of tracing paper, and the resulting points pricked through on to the drawing paper, so that no rubbing out will be necessary. This will apply to lines of camber and the outlines of girders with circular or elliptical top flanges, and to suspension bridges.

The positions of the rivets really require to be first determined before drawing the elevation of the girder, as the exact positions of the plate-joints will depend upon this. There must not be a flange-plate joint where the centre of a rivet comes, for that would necessitate a half-hole, which is no use at all in a tension-flange, and very objectionable anywhere. The rivets in the main angle-irons will be zigzagged in the two limbs. If there is a tee-iron stiffener in the centre of the span, there will be two rivets on each side of the centre in the vertical limbs of the main angle-bars, and therefore one rivet in the centre of the flange-plates; then the lengths of the plates to their joints, measured from the centre, will not be an even number of pitches, but will have a half-pitch in the length. If there is a plain bay in the centre of the span the centre line will fall between two rivets in the flange-plates, and the distances to the first joint on each side of the centre will be a number of whole pitches. It will be found most convenient to mark the centres of the rivets on the centre lines of the elevation first, and then draw

in the joints of the various plates, which should in every part fall midway between two rows of rivets.

In plate girders the setting out of the rivets is a very simple matter, but in triangular and lattice-webbed girders it is often far from easy. In the latter the shearing stresses, which in a plate girder are distributed over a considerable length of flange, are concentrated at certain points, and there is often a difficulty in finding room for the necessary number of rivets, unless a large joint-plate is used, which in many cases will give the girder a clumsy appearance. This may be avoided by using pin connections between the web-bars and flanges, but then there is a greater loss of sectional area in the flange than if rivets are used, and to meet this, additional thicknesses of plate are required at the joints. Pin connections cannot be conveniently adopted unless the flanges are trough-shaped; or the top flange may be trough-shaped and the bottom flange consist of bars, so that the connecting pins may be in double shear.

The length of bearing allowed to the main girders will depend upon the load they have to carry and the strength of the supporting piers. The load which comes upon the bed-plates of the girder may be spread over the substructure of the piers by substantial bed-stones. With 8 as a factor of safety, Bramley Fall sandstone will take 48 tons per square foot; limestone, 60 tons; Portland oolite, 33 tons; Derby grit, 25 tons; red Cheshire sandstone, 17 tons; and Yorkshire paving, 46 tons; blue Staffordshire brick will carry 17 tons, and ordinary stock brick about 5 tons.

These figures serve to determine the size of the bed-plates; the areas and thicknesses of the bed-stones must be proportioned to the bearing capacities of the sub-structures.

The main girders may be supported upon rollers or sliding plates, but for any span over 30 feet there should be a rocking bearing in the centre of the bed-plate, otherwise the whole weight will come upon the front edge of the pier as soon as the load is sufficient to deflect the girders.

Parts of the work that are to be turned, planed, or otherwise machined, should be indicated on the working drawings by red lines drawn parallel to the surfaces to be so treated, and a note should be made on each sheet of drawings in explanation thereof.

CHAPTER VIII.

PLATE GIRDER BRIDGES.

A PLAN of a steel plate girder bridge is shown in Fig. 59. It is intended to carry a double line of railway and to have a clear span of 80 feet, and width between girders 25 feet.

The floor consists of cross girders covered with plates of light trough flooring; they are spaced four feet apart and are well suited for buckled plates as a covering which will also form a very effective lateral bracing to the floor. Fig. 60 is a longitudinal section of the bridge, and Fig. 61 a half-transverse section. *A A* and *B B* are the main girders; *C C*, &c., the cross girders; and *DD*, a distributing girder. The whole maximum load will be taken as $1\frac{1}{2}$ tons per foot of span for each line of railway, so this will be the load upon each main girder as there are two main girders, and two lines of railway. The maximum load on a cross girder will occur when two locomotives are standing over it with their heaviest loaded wheels immediately over the cross girder; this I shall take as 16 tons per pair of driving wheels. The maximum load on one cross girder will be 32 tons, and it is so applied that it may be regarded as equally distributed.

The wheel base will span two bays of flooring, so that

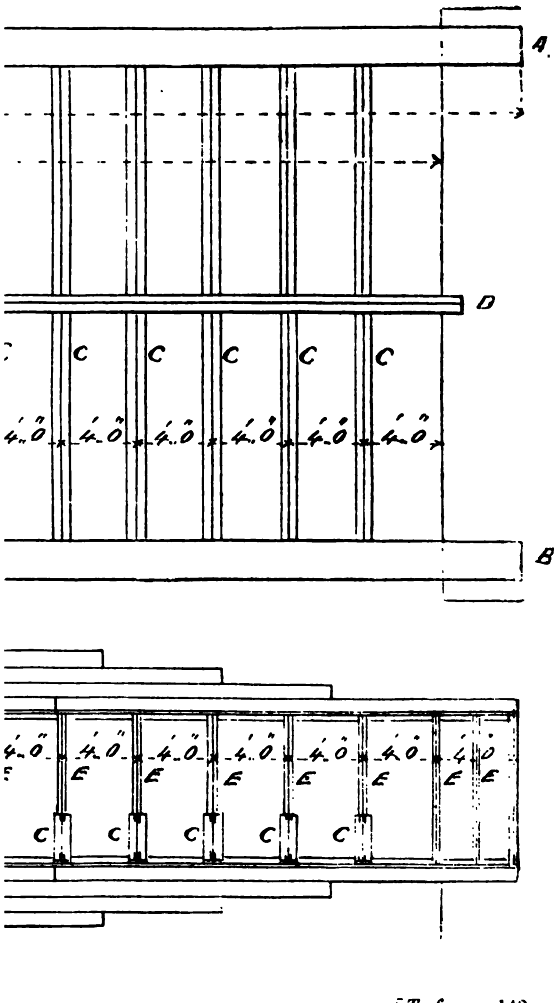
the alternate cross girders will carry no load, but weight of the floor, unless a distributing girder is used convey part of the load from the active to the idle gird The distributing girder will act with 8 feet effective s and under a central load; it will also put a central l upon the idle cross girder. The main girder flange-pla for this span will be made 2 feet wide, and the cross gir ends will be fixed to the webs of the main girders by riv and angle-steels, as shown in detail in Fig. 62; the effect span of the cross girders will, therefore, be 27 feet, distance from centre to centre of the main girders. T maximum stress on the loaded girder when part of t load is thus picked up at the centre will occur somewh between that point and each end of the girder, so the will be two maximum moments of stress.

Let l = span of girder = 27 feet; w = load per line foot; R = reaction at one end point of support; P = upward sustaining force at the centre of the girder; M = moment of stress at any point distant x from the neare end support.

At each end one-half the total load, less one-half th central force will act; therefore,

$$R = \frac{w\,l}{2} - \frac{P}{2}. \quad M = \frac{w\,.\,x^2}{2} - R\,x = \frac{w\,x^2}{2} - \frac{w\,l\,x}{2} + \frac{P}{2}$$

The points of maximum stress must now be determine When the moment of maximum stress is reached and about to commence diminishing, we may imagine an i definitely small increase of x during which the momen remains constant; here then, the increase of the positiv quantity must equal that of the negative quantity; let the x become $x + h$; h being indefinitely small:



[To face p. 142.

$$M = \frac{w}{2}(x+a)^2 + \frac{P}{2}(x+a) - \frac{w\,.\,l}{2}(x+a)$$

$$= \frac{w}{2}(x^2 + 2\,h\,x + h^2) + \frac{P}{2}(x+h) - \frac{w\,l}{2}(x+h).$$

The quantity h being originally very small compared to x, h^2 is so much smaller that it may be neglected; then the increase of the positive quantity is $w\,h\,x + \frac{P\,h}{2}$; and that of the negative quantity is $\frac{w\,h\,l}{2}$. Equating these

$$w\,h\,x + \frac{P\,h}{2} = \frac{w\,l\,h}{2};\ \text{therefore } x = \frac{l}{2} - \frac{P}{2\,w}$$

which is the value of x corresponding to the maximum moment of stress.

It will be seen that every cross girder will act alternately as an active and an idle girder, we must, therefore, ascertain under what circumstances the maximum stress will accrue, as all the cross-sections will have to be alike. Let $W = w\,.\,l$. If the sections are alike the maximum moments should be equal, therefore;

$$M = \frac{w}{2}\left(\frac{l}{2} - \frac{P}{2\,w}\right)^2 - \frac{w\,l}{2}\left(\frac{l}{2} - \frac{P}{2\,w}\right) + \frac{P}{2}\left(\frac{l}{2} - \frac{P}{2\,w}\right)$$

$$= \frac{1}{4}\left(P\,l - \frac{w\,l^2}{2} - \frac{P^2}{2\,w}\right) = -\frac{P\,l}{4}$$

whence,

$$\frac{P\,.\,l}{4} - \frac{W\,l}{8} - \frac{P^2\,l}{8\,W} = -\frac{P\,l}{4}$$

$$\frac{P^2}{8\,W} - \frac{P}{2} = -\frac{W}{8};\ P^2 - 4\,P\,.\,W + 4\,W^2$$

$$= 4\,W^2 - W^2 = 3\,W^2$$

$$P = 0{\cdot}268\,W.$$

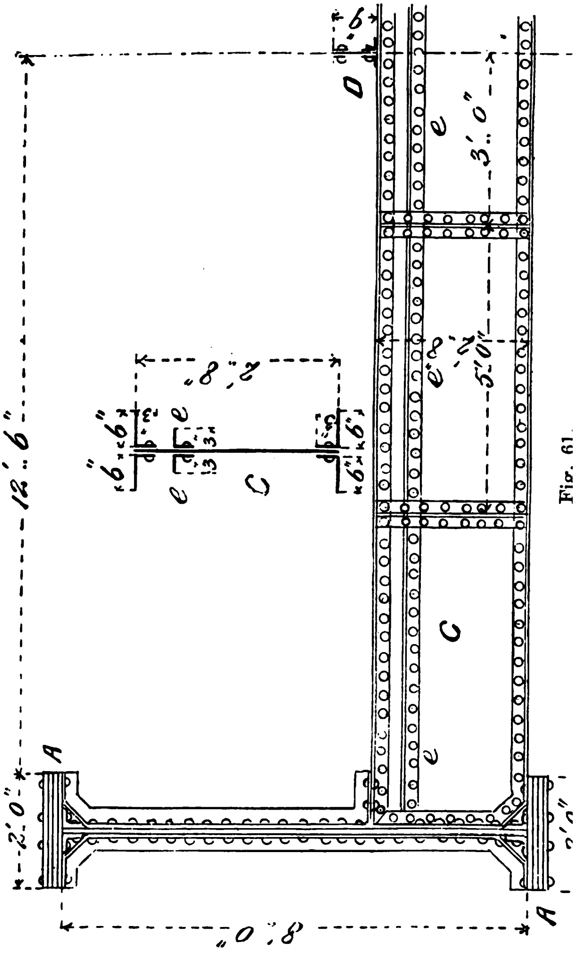


Fig. 61.

The distributing girder must have a deflection equal to the difference between the deflections of the active and idle girders to secure this proportion.

The load on the active girder will be $= W - 0{\cdot}268\ W = 0{\cdot}732\ W$;

and the central deflections will therefore be

$$D = 0{\cdot}732\ W \times \frac{l^3}{76{\cdot}8\ E\ I}$$

and

$$D_1 = 0{\cdot}268\ W \times \frac{l_1^3}{48\ E\ I}.$$

in which l_1 represents the effective span of the distributing girder, which is the distance between two alternate cross-girder centres—8 feet in the example—and D_1 = deflection of idle girder.

The deflection of the distributing girder will be in this case—

$$D_2 = 0{\cdot}732\ W \times \frac{l^3}{76{\cdot}8\ E I} - 0{\cdot}268\ W \times \frac{l^3}{48\ E I}$$
$$= 0{\cdot}00395 \times \frac{W.l^3}{E.I};$$

but applying the formulæ for deflection directly to the distributing girder we find

$$D_2 = 0{\cdot}00558 \times \frac{W.l_1^3}{E I_1} = 0{\cdot}00395 \times \frac{W l^3}{E I_1}$$

in which I_1 is the moment of inertia of the distributing girder; therefore, as the material is taken of the same quality throughout,

$$\frac{I}{I_1} = 0{\cdot}708\,\frac{l^3}{l_1^3},$$

in the example $l = 27$ feet; and $l_1 = 8$ feet; therefore

$$\frac{I}{I_1} = 0{\cdot}708 \times \left(\frac{27}{8}\right)^3 = 27{\cdot}217;\ I_1 = 0{\cdot}0367\ I.$$

This moment of inertia would be practically unattainable because the size of the girder would not allow proper connections to be made with the cross-girder, and we must therefore conclude that the distributing girder will take up more than the above proportion of the load and transfer it as a central load to the idle girder, and this will give the maximum stress upon any cross-girder. The maximum load picked up from the active girder cannot exceed that which would be taken off by an absolutely rigid distributing girder, which would make the deflections of the cross-girders equal, then,

$$\frac{W l^3}{76{\cdot}8\,E.I.} - \frac{P l^3}{48\,E I} = \frac{P l^3}{48\,E I};\ P = 0{\cdot}3125\ W.$$

Having disposed of this question, we can now proceed with the calculation of the section of the cross-girder, taking as a central load 32 tons $\times$ 0·3125 = 10 tons for the live load, to which is to be added the permanent weight of the flooring, ballast, and distributing and cross-girders themselves, which is equal on all the cross-girders.

The weight of ¼-inch buckled plates, including joint-strips and rivets, is 12 lbs. per superficial foot; ballast 1 foot thick, 120 lbs. per superficial foot; permanent way 400 lbs. per yard of double line. The area of floor carried by each cross girder will be 27 feet $\times$ 4 feet = 108 square feet.

Let the distributing girder be 9 inches deep with web-

plates $\frac{1}{4}$-inch thick, and four angle-steels 3 inches $\times$ 3 inches $\times$ $\frac{1}{2}$-inch thick to form the flanges.

A bar of iron 1 inch square and 1 yard long weighs 10 lbs., and from this fact the weights of iron sections are easily obtained by multiplying the sectional area by $3\frac{1}{3}$ to give the weight in lbs. per foot. It is evident that a square foot of wrought iron will weigh also 10 lbs. for every $\frac{1}{4}$-inch of thickness; mild steel weighs about 2 per cent. more. The four angle-steels of the distributing girder will weigh $9{\cdot}35 \times 4 = 37{\cdot}4$ lbs. per foot run; the 9-inch $\times$ $\frac{1}{4}$-inch web 7·66 lbs., and 5 per cent. must be allowed for rivet heads; this will make the weight of the distributing girder $= \{37{\cdot}4 + 7{\cdot}66\} \times 1{\cdot}05 = 47{\cdot}31$ lbs. per foot run; of this there will be 4 feet carried on the centre of each cross-girder $= 47{\cdot}31 \times 4 = 189{\cdot}24$ lbs., which is so trifling it may be neglected.

The equally distributed load will be—

Floor plates . .	12 lbs.	×	108 sq. ft.	=	1,296 lbs.
Ballast . . .	120 ,,	×	108 ,,	=	12,960 ,,
Permanent way .	400 ,,	×	$1\frac{1}{3}$,,	=	533 ,,
					14,789 ,,

which is equal to 6·6 tons; the effect of 10 tons central live load is equal to 20 tons equally distributed, so to estimate the weight of the cross-girder we have for the useful load $6{\cdot}6 + 20 = 26{\cdot}6$ tons. The section will be uniform throughout, and the depth of the cross-girders between the flange-plates, 2 feet 3 inches—one-twelfth of the span; the factor for the weight of the cross-girder from table (page 81) is 0·00243, therefore, the weight of the girder will be,

$$= 26{\cdot}6 \times 0{\cdot}00243 \times 27 = 1{\cdot}745 \text{ tons.}$$

The total weight to be provided for will be equivalent to a uniformly distributed load of $26{\cdot}6 + 1{\cdot}745 = 28{\cdot}345$ tons. We will see what the most economical depth would be; from the formulæ $d = \sqrt{\frac{w\, l^2}{48\, t\,.\, s}}$ page 88.

The working stress must now be determined, and I shall use as factor of safety 5, and specify the steel to have a tensile strength between 28 and 32 tons per sectional square inch, the lower of the two will be taken for calculating strengths, and the working stress will therefore be $= 28 \div 5 = 5{\cdot}6$ tons per sectional square inch. To find the value of t an allowance must be made for the web-stiffeners, which for girders of this size would be 6 inches $\times$ 3 inches $\times \frac{1}{2}$ tee-steels on both sides at 4 feet apart, centre to centre. The sectional area of two stiffeners will be $= 2\ \{6 + 3 - \frac{1}{2}\}\frac{1}{2} = 8{\cdot}5$ square inches, and this averaged along 4 feet will give $8{\cdot}5 \div 48$ inches $= 0{\cdot}177$ inch.

To determine the web area, the load on the active cross-girder is to be taken; live load $= 22$ tons, and dead load $6{\cdot}6 + 1{\cdot}745 = 8{\cdot}345$ tons, making, together, $8{\cdot}345 + 22 = 30{\cdot}345$ tons; half of this, $15{\cdot}172$ tons, is carried at each end of the cross-girder.

The resistance of steel to shearing stress is four-fifths of its tensile resistance, therefore its working stress per sectional square inch will be $= 5{\cdot}6 \times \frac{4}{5} = 4{\cdot}48$ tons; therefore the vertical sectional area of the web at each end must not be less than $15{\cdot}172 \div 4{\cdot}48 = 3{\cdot}386$ square inches. This would be satisfied by a thickness of $3{\cdot}386 \div 27$ inches depth $= 0{\cdot}125$ inch, for solid plate, but there are rivet-holes to be deducted where the web-plate is riveted to the vertical and angle-steels. The rivets will be in double shear as the angle-steels will be upon both sides of the web, and

they will be $\frac{3}{4}$ inch in diameter, which gives a sectional area of 0·44 square inches, or 0·88 square inch for each rivet in double shear; the rivet area must be not less than 3·386 square inches, therefore the number of rivets required will be $= 3{\cdot}386 \div 0{\cdot}88 = 4$ rivets. This gives a pitch of about 7 inches, but in order to make the work more solid and give more continuous support to the end edges of the web-plates I should use a 4-inch pitch, and that will give 7 rivets at each end of the web, thus reducing the effective height of the web at that place to $27 - 7 \times \frac{3}{4} = 21{\cdot}75$ inches. The web thickness required will then be $3{\cdot}386 \div 21{\cdot}75 = 0{\cdot}156$ inch. The plates would not be made less than a quarter of an inch thick, so that the average, including the stiffeners, will be $t = 0{\cdot}25 + 0{\cdot}177 = 0{\cdot}427$ inch. The weight of the maximum load per foot run is $w = 30{\cdot}345 \div 27 = 1{\cdot}124$ tons. Filling in these figures in the formulæ for economical depth, we get

$$d = \sqrt{\frac{w \,.\, l^2}{48\, t \,.\, s.}} = \sqrt{\frac{1{\cdot}124 \times (27)^2}{48 \times 0{\cdot}427 \times 5{\cdot}6}} = 2{\cdot}672 \text{ feet.}$$

This is 2 feet 8 inches in depth; by using this, the ratio of span to depth is nearly 9, and the weight of the girder will then be

$$= 26{\cdot}6 \times 0{\cdot}00198 \times 27 = 1{\cdot}422 \text{ tons.}$$

The loads from which we must determine the amount of stress at the centre are a central live load of 10 tons, and an equally distributed load of $6{\cdot}6 + 1{\cdot}422 = 8{\cdot}022$ tons. The whole moment of stress at the centre will, therefore, be

$$\frac{10 \times l}{4} + \frac{8{\cdot}022 \times l}{8} = \frac{10 \times 27}{4} + \frac{8{\cdot}022 \times 27}{8}$$

$$94{\cdot}574 \text{ tons.}$$

Let $a =$ the sectional area of one flange in square inches, then the moment of resistance will be

$$M = s.\ a.\ d = 5{\cdot}6 \times a \times 2{\cdot}67 = 14{\cdot}95\ a$$

As the moments of stress and resistance must be equal, therefore,

$$M = 14{\cdot}95\ a = 94{\cdot}574$$

And

$$a = \frac{94{\cdot}574}{14{\cdot}95} = 6{\cdot}32 \text{ square inches.}$$

The flange area required can be furnished by angle-steels without requiring any plates; if two angle-steels 6 inches $\times$ 3 inches $\times$ $\frac{5}{16}$ inch thick are used for the flange, with the longer limbs horizontal, these will give an area of 6·75 square inches. There will be no rivet-holes in the bottom flanges, and those in the top flange for the connections with the distributing girders will not diminish the resistance to compression. The floor plates may be riveted to the top angle-steels, or to save thickness of floor they may be carried by light angle-steels *e e*, Fig. 61, riveted on to the web of the girder between the stiffeners.

We are sometimes tied in our heights, so that we cannot get room for cross-girders of the proper depth, in which case we must make the best of what we can get; but if the girder is shallower it will be proportionately heavier.

The rivets connecting the distributing girder with each cross-girder will carry 10 tons, and will be in tension, as the bottom flange of the distributing girder will be riveted to the top flange of the cross-girder; the rivet area required will be 10 tons $\times$ 5·6 $=$ 1·786 square inches. The sectional area of a $\frac{7}{8}$-inch rivet is 0·6 square inches; so four of these will give a total sectional area of $0{\cdot}6 \times 4 = 2{\cdot}4$ square inches.

The main girders have now to be designed. The live load upon one will be 1·5 tons × 80 feet span = 120 tons; the area of half the floor, measuring it from centre to centre of the main girders, is = 80 × 13·5 = 1,080 square feet. The total superincumbent load on one main girder will be—

Floor plates .	12 lbs. × 1,080 sq. ft.	= 12,960 lbs.	
Ballast . .	120 ,, × 1,080 ,, ,,	= 129,600 ,,	
Permanent way . .	400 ,, × 27 yds.	= 10,800 ,,	
		153,360 ,,	= 68,464 tons.
Live load .	1·5 tons × 80 ft.		= 120,000 ,,
			188,464 ,,

The depth of the girder may be assumed as 8 feet in a trial depth, then, as the plates will be proportioned to the varying stress, the factor for girder weight per foot will be 0·0017, and the dead weight of the girder will be

$$= 188{\cdot}464 \times 0{\cdot}00167 \times 80 = 25{\cdot}176 \text{ tons.}$$

The total load will then be 188·464 + 25·176 = 213·64 tons. We will now find the proper depth of girder. The shearing stress over each supporting pier will be 213·64 ÷ 2 = 106·82 tons, which at a working stress of 4·48 tons per sectional square inch, will require 106·82 ÷ 4·48 = 24 square inches nearly. Just over the pier there will be a pair of web-stiffeners, and therefore a row of rivets; if these are 4 inches pitch there will be 24 of them in the vertical height of the web, causing a loss of effective height = 24 × ¾ = 18 inches, and the effective height left is 96 – 18 = 78 inches. The thickness of web required will, therefore, be

$$= \frac{24}{78} = 0{\cdot}3 \text{ inch.}$$

The commercial thickness would be $\frac{3}{8}$ inch, and this would be kept throughout the length of the girder. The tee-steel stiffeners I should make 6 inches × 4 inches × $\frac{3}{4}$ inch thick, and place them 4 feet apart. The sectional area of each pair will be 13·875 square inches, which, spread over 48 inches of length, shows an average additional thickness $= 13{\cdot}875 \div 48 = 0{\cdot}289$; therefore the effective thickness of web, including allowance for stiffeners, will be $= 0{\cdot}375 + 0{\cdot}289 = 0{\cdot}664$ inches. The load per foot run $= 213{\cdot}64 \div 80 = 2{\cdot}67$ tons $= w$; and $s = 5{\cdot}6$ tons.

$$d = \sqrt{\frac{w \, . \, l^2}{48 \, t \, . \, s.}} = \sqrt{\frac{2{\cdot}67 \times (80)^2}{48 \times 0{\cdot}664 \times 5{\cdot}6}} = 9{\cdot}784 \text{ feet.}$$

If we ran to this height it would necessitate heavier web-stiffeners than those taken, and the web itself would be preferred of thicker metal; so for this case we will keep the 8 foot depth.

The sectional area at the centre of the span for the bottom flange, taken nett, must not be less than

$$\frac{W l}{8 \, s \, . \, d} = \frac{213{\cdot}64 \times 80}{8 \times 5{\cdot}6 \times 8} = 47{\cdot}7 \text{ square inches.}$$

The angle steels used will be 4 inches × 4 inches × $\frac{1}{2}$ inch; the area of the horizontal limbs will be, deducting rivet-holes, 3·125 square inches, leaving—to be made up in the plates—$47{\cdot}7 - 3{\cdot}125 = 44{\cdot}575$ square inches. There will be four rows of $\frac{7}{8}$-inch rivets in each flange, and this will reduce its effective width by $0{\cdot}875 \times 4 = 3{\cdot}5$ inches, leaving $24 - 3{\cdot}5 = 20{\cdot}5$ inches of effective width. The total thickness of the flange at the centre of the span will be $44{\cdot}575 \div 20{\cdot}5 = 2{\cdot}174$ inches. The same thickness will be kept for the plates of the top flange.

As this thickness is something more than ⅛ inch, and working to sixteenths is too minute for ordinary work of a heavy character, I shall make the flanges 2¼ inches thick, the two inner tiers will be ½ inch thick, and the outer ones each ⅝ inch thick.

The solid resistance of the steel to compression exceeds its tensile strength, but on account of its liability to cripple it is better to keep in the top flange, the margin which is given by keeping it of the same gross area as the tension flange.

The next step is to see at what points the flange-plates can be reduced to suit the diminishing stress.

The formulæ arrived at (page 94) for the length of each tier of plates is

$$L = 2\sqrt{\frac{l^2}{4} - \frac{2\,.\,s\,.\,A\,.\,d}{w}}$$

In which $L =$ the length of the tier of plates, and $A =$ the sectional area of the flange beyond the tier under consideration: for any one case this expression may be condensed, for the present one

$$L = 2\sqrt{\frac{(80)^2}{4} - \frac{2 \times 5{\cdot}6 \times A \times 8}{2{\cdot}67}}$$

$$= \sqrt{6400 - 134{\cdot}24\,A}$$

The effective sectional area of the bottom flange at the centre of the span will be: angle-steels, 3·125 square inches; plates, 20·5 × 2·25 = 46·125; total, 46·125 + 3·125 = 49·25 square inches. The reduced sectional areas, as the plates are stopped, will be: 49·25 — 20·5 × ⅝ = 36·44 square inches; 36·44 − 20·5 × ⅝ = 23·63 square inches;

and $23{\cdot}63 - 20{\cdot}5 \times \frac{1}{2} = 13{\cdot}38$ square inches; the inner tier of plates runs the whole length of the girders; the lengths of the other tiers, commencing with the outside ones, will be:

Outer tier, $L = \sqrt{6400 - 134{\cdot}24 \times 36{\cdot}44} = 38{\cdot}8$ feet;

2nd „ $L = \sqrt{6400 - 134{\cdot}24 \times 23{\cdot}63} = 56{\cdot}8$ „ ;

3rd „ $L = \sqrt{6400 - 134{\cdot}24 \times 13{\cdot}38} = 67{\cdot}8$ „ ;

The next lengths above these that will suit the pitch of the rivets will be adopted; there will be a stiffener at the centre of the span, so there will be a rivet on each side of the centre, and this will give an even number of pitches to the rivets in each tier of plates.

We must now see what pitch of rivets will suit; with $\frac{7}{8}$ inch diameter, 5 inches is suitable as far as workmanship is concerned; with the first pair of rivets 5 inches from the bearing, the stress upon them would be

$$= \frac{w \, . \, x^2 - w \, . \, l \, . \, x}{2 \, . \, d} = \frac{2{\cdot}67 \times (0{\cdot}416)^2 - 2{\cdot}67 \times 80 \times 0{\cdot}416}{2 \times 8}$$
$$= 5{\cdot}52 \text{ tons.}$$

The shearing resistance of two $\frac{7}{8}$-inch rivets, is $2 \times 0{\cdot}6$ square inches $\times$ $4{\cdot}48$ tons $= 5{\cdot}376$ tons; this is not quite sufficient, as we do not consider joint friction, therefore, for the first few rivets, the pitch may be kept down to 4 inches.

Next come the cover-plates for the flanges, and these will be the same for both top and bottom flanges, so they will be calculated for the bottom flange. The effective working resistance of one $\frac{5}{8}$-inch plate will be $20{\cdot}5 \times \frac{5}{8} \times 5{\cdot}6 = 71{\cdot}747$ tons; the rivet area to resist this stress, on each

side of the joint, will be $71 \cdot 747 \div 4 \cdot 48 = 16$ square inches, and, therefore, the number of $\frac{7}{8}$-inch rivets required will be $16 \div 0 \cdot 6 = 26 \cdot 6$—that is 27 as we cannot have a fraction of a rivet, and, in fact, as there are four rows of rivets, this will have to be 28, making 7 rivets in each row, and the length of each lap of the cover-plate 7×5 inches pitch $= 35$ inches. For the $\frac{1}{2}$ plates, the working resistance will be $20 \cdot 5 \times \frac{1}{2} \times 5 \cdot 6 = 57 \cdot 4$ tons; the rivet area to resist this will be $57 \cdot 4 \div 4 \cdot 48 = 12 \cdot 81$ square inches, and, therefore, the number of rivets required in each lap of cover-plate will be $12 \cdot 81 \div 0 \cdot 6 = 21$ rivets, which, as there are four rows, will become 24, giving to each lap a length of $6 \times 5 = 30$ inches.

The length of the bed-plate and bearing at each end will be fixed at 4 feet, then the total length of main girder will be 88 feet. In iron this would have to be divided into five lengths, but in steel it can easily be rolled in three lengths; the next tier and that outside it, can both be in two lengths, and the outside tiers in one length. The joints in the second and third tiers from the inside can be brought under one cover-plate in the centre, and for uniformity, the three laps of cover-plate will be made equal, as shown in Fig. 60. This will make the length of the central cover-plate 35 inches $\times 3 = 105$ inches $= 8$ feet 9 inches.

The joints of the inner tiers of plates may be brought under the ends of the outer tiers of plates, so that these being lengthened, form the cover-plates; the fact that they are thicker than the inner plates does not matter, in fact gives a better hold to the rivets; the length to be added will be 2 feet 6 inches at each end.

There are yet the joint-plates of the web, and angle-steel joints to be disposed of. Over the pier is the heaviest stress upon the web, where it amounts to half the total

load on the girder, that is 106·82 tons, and the rivet area to take this load will be $106·82 \div 4·48 = 24$ square inches nearly; therefore, the number of rivet areas required $= 24 \div 0·6 = 40$.

These rivets are in double shear, so only 20 will be required on each side of the joint. The depth of the web being 96 inches, a single row of rivets will be sufficient, and the pitch will be $96 \div 20 = 4·3$ inches. The centre lines of rivets in the limbs of angle and tee-bars run in the centre of the clear transverse width. In a 4 inch $\times$ 4 inch $\times$ $\frac{1}{2}$-inch angle-bar, the clear width is $3\frac{1}{2}$ inches; therefore, the centres of the rivets will be $1\frac{3}{4}$ inches in from the edge, and $2\frac{1}{4}$ inches from the back, this will bring the lines of rivets connecting the main angle-steels with the web, $2\frac{1}{4}$ inches in from the top and bottom, and the space between these must be divided up for 18 rivets, that is, into 19 spaces. The tee-steel stiffeners will supply the web-covers in this case. In shallow girders which are heavily loaded, it is frequently necessary to put wider cover-plates on the webs between them and the tee-stiffeners, so that there may be room for two or more rows of rivets on each side of the joint.

The tensile resistance of the angle-steel is equal to its nett sectional area, multiplied by 5·6 tons. The rivets in the angle-steels are zigzag, so only one will be deducted from the mean width of the two limbs, leaving the effective width $7·5 - 0·875 = 6·625$ inches, the tensile strength will be $6·625 \times \frac{1}{2} \times 5·6 = 18·55$ tons, which will require in shearing area, $18·55 \div 4·48 = 4·14$ square inches, and the number of $\frac{7}{8}$-inch rivets to make up for this area is $4·14 \div 0·6 = 7$ rivets; the odd number will suit here, as the rivets are not opposite each other; there will be three joint-rivets in one limb, and four in the other, on each

side of the joint. The rivets in the angle-covers will be in single shear. It may be mentioned that angle-bars for covers are rolled "round backed," to fit into the roots of the main angle-bars. If the edges of the limbs of the angle-covers are to lie flush with those of the main angle-bars, they should be made thicker in proportion, say $3\frac{1}{2}$ inches $\times$ $3\frac{1}{2}$ inches $\times$ $\frac{5}{8}$ inch. We will see how this will work out as to equality of strength. The nett sectional area of one main angle-steel is $= 6{\cdot}625 \times \frac{1}{2} = 3{\cdot}3125$ square inches. The nett width of the cover will be $6{\cdot}5 - 0{\cdot}875 = 5{\cdot}625$ inches, and its sectional area $5{\cdot}625 \times \frac{5}{8} = 3{\cdot}515$ square inches—showing a slight margin in favour of the cover-angle. The angle-steels can be obtained over 44 feet long, so one joint will be all that is absolutely necessary, though, for convenience of manufacture, it may be preferred to make the main angle-steels in three lengths.

It is very usual to make the angle-iron joints on each side of the web miss each other, "break-joint" as it is termed, so that two joints shall not come together; this is illogical if we follow the rule of making the joint as strong as the solid metal, for then it should not matter where the joints come; but still, it is advisable in view of possible defects in workmanship.

It is necessary to ensure sufficient horizontal sectional area in the stiffeners over the piers to resist the load carried through them to the bed-plates. The tee-steels on the front of the pier have each a sectional area of $4\frac{1}{4}$ square inches, 8·5 square inches together; in the length of bearing, there will be another pair at 2 feet from the edge of the pier and the end angle-irons; these latter have a gross area of 8 sectional square inches; the end-plate cannot be taken as well, because any stress upon it must come through the end angle-steels.

The total area then to resist this compression is,

4 Tee-steels	=	17	square inches
2 End angle-steels	=	8	,, ,,
		25	,, ,,

The load to be carried is 106·82 tons; so the stress square inch of section will be 106·82 ÷ 25 = 4·19 per sectional square inch. As this does not come on tops of the stiffeners, but gradually accrues towards t bottoms, the strength is ample.

CHAPTER IX.

TRIANGULAR AND LATTICE GIRDER BRIDGES.

As an example I shall take a triangular-web girder bridge, shown in elevation at Fig. 62, and in plan at Fig. 63. The bridge is assumed to have a clear span of 85 feet, and an effective girder span of 90 feet between the points 11 and 21. The web is made up by bars which, with the flanges *A B* and *C D*, form a series of equilateral triangles. The girders are each divided into ten triangles, the load is brought on to the bottom flanges at the points of connection with the web-bars, so that no transverse stress shall come upon the flanges. The bridge is required to carry a single line of railway, and the clear width between the flanges of the main girders will be 16 feet. The connections between the web-bars and the flanges of the main girders will be made by pins, which are more suitable than groups of rivets for this purpose, because they do not cut away so much sectional area in proportion to their strengths, and also they allow of deflection in the girders without incurring racking stress, which must occur when joint-plates are used to make these connections, because in the ordinary course of matters the deformation will be attended by changes of the angles of the joints—after deflection the girder's elements will still be rectilineal, and not curved, as in the plate girder.

The distances between the apices of the triangles formed

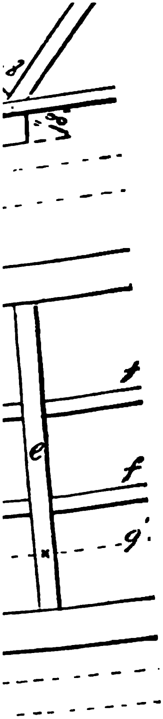
by the centre lines of the webs and those of the will be 9 feet, and that will also be the distance from to centre of the cross-girders *e e*, &c. This distance much to be spanned by floor plates or longitudinal sle and therefore these rail-bearing girders *f f* betwee cross-girders and the flooring between them, can be up with floor-plates, either buckled plates or the sh trough flooring described on page 79, and illustrat Fig. 50.

I shall not, in dealing with the details of this and fo ing examples, determine the economical depths of girders, the method has been sufficiently explained in previous chapter, and in this I shall use such depths a convenient in practice. The floor-girders will be with depth about one-twelfth of the span. The girders are divided into ten bays of 9 feet each; there the web-bars are also 9 feet long between centres. F any apex, say 5, let fall a vertical line 5 g upon the 15, 16 of the triangle, then the triangle 5 g 15 will b right-angled triangle, and 5 g, the depth of the main gir will be,

$$d = \sqrt{(5\,;15\,;)^2 - (15\,;g)^2} = \sqrt{9^2 - (4{\cdot}5)^2} = 7{\cdot}794 \text{ fe}$$

Fig. 64 shows a cross section of the bridge, and we m begin by determining the sizes of the floor-girders *f f* a *e e*. The spans of the former will be 9 feet, and the ma mum stress will occur when a locomotive wheel is at t centre of the span, its load being taken as 8 tons. The will be two kinds of load upon girders *f*, this central li load and the equally distributed load of the flooring, ballas and permanent way.

The material to be taken in this example is wroug iron, with a specified tensile resistance of 22·5 tons p


e
f
f
g

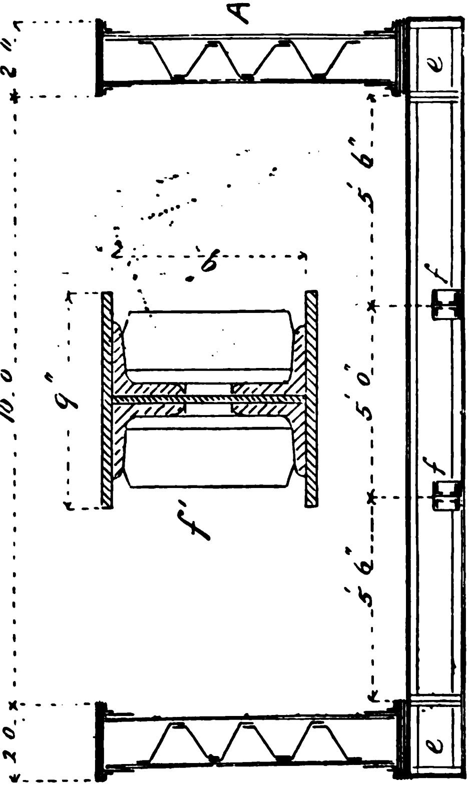


Fig. 64.

sectional square inch, and factor of safety 4·5, which gives a working tensile resistance of 5 tons per sectional square inch of nett area; and 19 tons crippling resistance, which gives for compression 4·2 tons per sectional square inch of gross area.

The floor-plates at the sides will be supported upon angle-iron riveted to the bottom plates of the main girders. The portion of floor carried upon one rail-bearer will be half that between it and the main girder, plus half that between it and the other rail-bearer, a total width of $2 \cdot 75 + 2 \cdot 5 = 5 \cdot 25$ feet, and the length is 9 feet; therefore the permanent load will be—

Floor plates .	12 lbs.	× 9 ft.	× 5·25 ft.	=	567 lbs.
Ballast .	. 120 ,,	× 9 ,,	× 5·25 ,,	=	5,670 ,,
Permanent way	200 ,,	× 3 yds.	.	. =	600 ,,
					6,837 ,, = 3·052 tons.

The central load of 8 tons will produce a stress equal to that from a uniformly distributed weight of 16 tons; so 19·052 will be the load to derive the weight of the girder from, its depth is to be one-twelfth the span, so its weight, the section being uniform throughout,

$$= 19 \cdot 052 \times 0 \cdot 00243 \times 9 \text{ feet} = 0 \cdot 417 \text{ ton.}$$

It may be well to mention here that these weights are for materials working at 5 tons per sectional square inch, the weight will vary inversely as this working strength, so where the strength differs notably from this an allowance must be made; if, for instance, this girder were to be made of special steel carrying a working stress of 7 tons per square inch, the weight of the rail-bearer would be $0 \cdot 417 \times \frac{5}{7} = 0 \cdot 298$ ton.

The permanent load including the weight of the rail-bearer, will be $3{\cdot}052 + 0{\cdot}417 = 3{\cdot}469$ tons. The depth between the flanges will be 9 inches as the span is 9 feet. The central moment of stress will be the sum of the moments due to the central and the equally distributed load, or

$$M = \frac{Wl}{4} + \frac{W'l}{8} = \frac{8 \times 9}{4} + \frac{3{\cdot}469 \times 9}{8} = 21{\cdot}9 \text{ foot tons.}$$

The moment of resistance is $= s \,.\, a \,.\, d$ and $s \,.\, a \,.$ is the direct resistance of the flange, a varying inversely as s. For the top flange; $d = 9$ inches $= 0{\cdot}75$ feet.

$$M = 21{\cdot}9 = s.a.d = 4{\cdot}2 \times a \times 0{\cdot}75$$

and the gross sectional area will therefore be,

$$a = \frac{21{\cdot}9}{4{\cdot}2 \times 0{\cdot}75} = 7 \text{ square inches (nearly).}$$

For the bottom flange the nett sectional area required will be,

$$a = \frac{21{\cdot}9}{5 \times 0{\cdot}75} = 6 \text{ square inches (nearly).}$$

It is useless to take fractions of inches into account in settling these sectional areas.

It is convenient in practice to make the top and bottom flanges alike in these smaller elements of the structure; so in this case if the angle-irons are $3'' \times 3'' \times \frac{1}{2}''$ and the flange-bars $9'' \times \frac{1}{2}''$ we shall have a suitable size. When the flange is small enough to require a bar only—under 12 inches wide, in addition to the angle-iron horizontal limbs the width should be kept to equal inches to suit the sizes of bars ordinarily rolled. Over that width the flanges are made of plates, which, having to be sheared along the

edges, can be supplied to any dimension within the rolling width of the mill.

In the top flange of our rail-bearer the sectional area will be, angle-iron limbs $2\{3'' \times \frac{1}{2}''\} = 3$ square inches, bar $9'' \times \frac{1}{2}'' = 4{\cdot}5$ inches, total $7{\cdot}5$ square inches. In the bottom flange the nett sectional area will be, angle-iron limbs $2\{3 - 0{\cdot}75\}\ \frac{1}{2}'' = 2{\cdot}25$ square inches. Bar, $\{9 - 1{\cdot}5\}\ \frac{1}{2} = 3{\cdot}75$ square inches. Total nett area $2{\cdot}25 + 3{\cdot}75 = 6$ square inches.

The deductions are made for $\frac{3}{4}$-inch rivets, which are suitable for the dimensions used. The shearing resistance is taken as $\frac{4}{5}$ of the tensile, that is 4 tons per sectional square inch. The whole load on the bearer is $8 + 3{\cdot}469 = 11{\cdot}469$ tons, so the shearing stress on the web through the centre line of each end row of rivets will be $5{\cdot}734$ tons; the rivet area required to pick this up will be $5{\cdot}734 \div 4 = 1{\cdot}433$ square inches. The area of a $\frac{3}{4}$-inch rivet is $0{\cdot}44$ square inch, so the number of rivet areas will be $1{\cdot}433 \div 0{\cdot}44 = 3{\cdot}2$, that is 4, and as the web rivets are in double shear two rivets will be sufficient, but as the web-plate is 9 inches deep we can get three rivets in the height; this will leave the effective height of the web-plate $9 - 3 + 0{\cdot}75 = 6{\cdot}75$ inches and the thickness to resist the shearing stress will be $= 1{\cdot}433 \div 6{\cdot}75 = 0{\cdot}212$ inches. Therefore one quarter of an inch will be sufficient for the thickness of the web-plate; it should have two stiffeners on each side of the web, 3 feet from each end, and these may be $6'' \times 3'' \times \frac{1}{2}''$ tee-iron, as shown on the enlarged cross-section f'; these ends rest against the vertical limbs of the angle-irons and have packing pieces between them and the web.

We will now turn to the cross-girders e; fixing the width of the main girder flanges at two feet, the horizontal distance between their ver[illegible]re lines will be 18 feet, and this

will be the effective span of the cross-girders, which are to be riveted or bolted up to the under side of the bottom flange, and which will be carried across so that the load on the flange is equally distributed on each side of the centre of the main girder. The maximum stress occurs on a cross-girder when a pair of driving wheels comes directly over the point of connection with the rail bearers, and at these points the permanent load also comes on the cross-girder. The weight of each wheel is 8 tons, and permanent weight will be half from each bearer meeting at the cross-girder—the total $8 + 3{\cdot}469 = 11{\cdot}469$ tons; the moment of stress will reach its maximum under the centre of the rail 6 feet 6 inches from the end of the effective span of the cross-girder. The reaction at each point of support is equal to one of the symmetrical loads, and if x passes beyond the first rail, the downward moment of the local load will just balance the increase of the negative moment, so the stress from these loads will be constant between the rails.

The maximum moment of stress will be, from the above load,

$$= 11{\cdot}469 \times 6{\cdot}5 = 74{\cdot}548 \text{ feet tons.}$$

To get at the weight of the cross-girders, the uniform load to produce an equal maximum stress must be found, which will be given by the following equation;

$$M = \frac{W.l}{8} = \frac{W \times 18}{8} = 74{\cdot}548; \ \therefore \ W = 74{\cdot}548 \times \frac{8}{18}$$
$$= 33{\cdot}13 \text{ tons.}$$

As the depth of the cross-girder is to be one-twelfth of its span, the girder weight will be,

$$= 33{\cdot}13 \times 0{\cdot}00243 \times 18 \text{ feet} = 1{\cdot}45 \text{ tons.}$$

This is an equally distributed load, which will give a maximum central moment of stress,

$$= \frac{W.l}{8} = \frac{1{\cdot}45 \times 18}{8} = 3{\cdot}262 \text{ tons.}$$

This, added to the moment for the concentrated load, gives the total moment—$74{\cdot}548 + 3{\cdot}262 = 77{\cdot}81$ feet tons. As the depth of web is one-twelfth the span it will be 18 inches = 1·5 feet. For the top flange

$$M = 77{\cdot}81 = s.a.d = 4{\cdot}2 \times a \times 1{\cdot}5\,;$$

the gross sectional area will therefore be,

$$a = \frac{77{\cdot}81}{4{\cdot}2 \times 1{\cdot}5} = 13 \text{ square inches (nearly).}$$

For the bottom flange the nett sectional area will be

$$a = \frac{77{\cdot}81}{5 \times 1{\cdot}5} = 11 \text{ square inches (nearly).}$$

There will be two rows of ¾-inch rivets in each flange, and angle-irons 3 inches × 3 inches × ½-inch will be used to connect the web with the flanges. The gross area of the horizontal limbs of the angle-irons is $\{3 + 3\}\,\frac{1}{2} = 3$ square inches; this leaves, to be made up in plates, for the top flange $13 - 3 = 10$ square inches. For the bottom flange the angle-iron section is $2\,\{3 - 0{\cdot}75\}\,\frac{1}{2} = 2{\cdot}25$ square inches, leaving for the plates $11 - 2{\cdot}25 = 8{\cdot}75$ square inches. So if two plates ½-inch thick are used, their least width, adding the loss by rivet holes, must be $8{\cdot}75 + 1{\cdot}5 = 10{\cdot}25$ inches. In practice each flange will have two 12-inch × ½-inch plates which give a slight margin of

[illegible]

The maximum shearing stress upon the web will be,

$$11{\cdot}469 + \frac{1{\cdot}45}{2} = 12{\cdot}194 \text{ tons.}$$

The vertical sectional area of the web at the point of support must therefore be not less than $12{\cdot}194 \div 4 = 3{\cdot}048$ square inches. The depth of the web will be 18 inches, and if 3 inches × 3 inches × ½-inch angle-irons are used, the distance of each end rivet from the edge of the web—in a vertical row—will be 1¾ inches. To keep the horizontal lines of rivet-centres in the middle of the clear width of the vertical limbs, the distance between will be $18 - 1{\cdot}75 \times 2 = 14{\cdot}5$ inches. The number of rivets used to fasten the stiffeners, in addition to those passing through the angle-irons, would be two; so the loss of depth in the web-plate will be four rivet-holes, and the nett depth will be $18 - 0{\cdot}75 \times 4 = 15$ inches, whence the web thickness is found to be $3{\cdot}048$ square inches $\div\ 15 = 0{\cdot}203$ inches, so ¼-inch will be amply thick.

In riveting or bolting the cross-girder to the bottom flange of the main girder, eight rivets or bolts would be used, as there will be four rows of rivets in the horizontal plates of the bottom flange. Eight ¾-rivets give a sectional area $0{\cdot}44 \times 8 = 3{\cdot}52$, and the tensile resistance being 5 tons per sectional square inch this is equal to a load $= 3{\cdot}52 \times 5 = 17{\cdot}6$ tons. The actual load is 12·194 tons, so there is a good margin here to allow for defective riveting.

If bolts are used, the sectional area—when in longitudinal stress—must be measured at the bottom of the thread; defects that occur in rivet-heads are absent in bolts, so nett areas will be safe. The sectional area required will be $12{\cdot}194 \div 5 = 2{\cdot}439$ square inches, and this,

divided by the number of bolts 8, gives $2{\cdot}439 \div 8 = 0{\cdot}304$ inches as the sectional area of each bolt. This corresponds to a diameter of 0·622 inch, and a bolt $\frac{3}{4}$-inch diameter (Whitworth's Standard) has at the bottom of the thread a diameter of 0·656 inch. So this is sufficient for the purpose.

We now come to the main girders. The loads will be summed for each point at which it accrues; but the permanent load only will be taken from the direct cross-girder loads; the running loads would not be fairly represented by a maximum load pair of driving wheels on each cross-girder, so the running load is taken at per foot span and the apex load taken for that. I some time since worked out a table of uniformly distributed loads equivalent to a test load of goods' locomotives, the moments of stress from the latter being determined by taking the load on each pair of wheels as a concentrated load. There is no averaging; and the engines were all headed in one direction. Some people had an idea that the greatest stress occurred when the train of engines was arranged like two trains meeting at the centre of the bridge; but I found by calculating each way that the maximum stress occurs when the engines follow each other, the chimney of one next to the tender of that preceding it.

The class of engine taken is heavy goods, and the exact wheel loads and wheel bases of engine and tender are shown in diagram in Fig. 65. *A* shows the position of the front buffer of the engine, and *B* that of the back buffer of the tender. The total, equally distributed loads for different spans, are given in the following table, for one track of railway :—

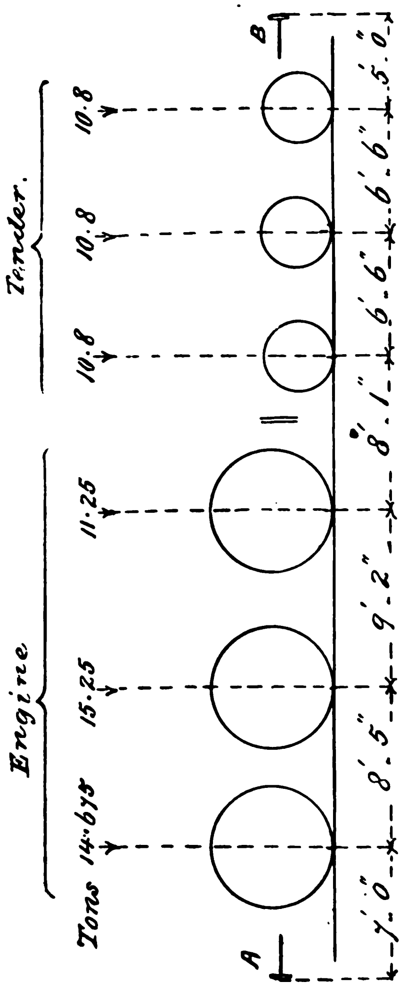

Engine
Tender.
Tons 14·675
15·25
11·25
10·8
10·8
10·8
A
B
7' 0"
8' 5"
9' 2"
8' 1"
6' 6"
6' 6"
5' 0"

Fig. 65.

Span in feet.	Load in tons.	Span in feet.	Load in tons.
16	31	60	90
18	32	65	98
20	37	70	105
22	41	75	113
24	45	80	119
26	48	85	125
28	50	90	133
30	53	95	140
32	54	100	145
34	56	105	153
36	58	110	161
38	60	115	169
40	63	120	175
42	66	125	183
44	68	130	190
46	71	135	198
48	73	140	205
50	78	145	212
55	83	150	219

We have here got below 1·5 tons per lineal foot of span, and all spans above 150 feet may be taken as having that as an outside load.

For our example 90 feet effective span, the test load is 133 tons, which gives $133 \div 90 = 1{\cdot}47\dot{7}$ tons.

The running load at each point of junction of the web-bars with the lower flange will be $= 1{\cdot}47\dot{7} \times 9 = 13{\cdot}3$ tons. The weight of rail-bearers and their permanent load on each cross-girder is $3{\cdot}469 \times 2 = 6{\cdot}938$ tons; the weight of the cross-girder is 1·45 tons, so the total useful load at each apex of a triangle in the bottom flange will be,

$$\frac{13{\cdot}3 + 6{\cdot}938 + 1{\cdot}45}{2} = 10{\cdot}844 \text{ tons};$$

this is for one main girder.

We find from practice that for any span over 80 feet, a triangular girder will be lighter than a plate-girder for *equal* loads, therefore, in estimating the weight of the

main girders for the purposes of calculation as if they were plate-girders, we shall be on the safe side; if it is worth while, a reduction in sectional areas can be made if this estimate is much over what the girder works out to when detailed. The effective depth of the girder is 7·794 feet, so the ratio of span to depth is $90 \div 7{\cdot}794 = 11{\cdot}5$; the nearest in the table is 12, and as the sections will be proportioned to the various stresses, the tabular factor is 0·00189, and the estimated weight of one main girder will be,

$$10{\cdot}844 \times 9 \text{ (points of load)} \times 0{\cdot}00189 \times 90 \text{ feet span}$$
$$= 16{\cdot}6 \text{ tons.}$$

This weight will not be absolutely equally distributed, but it will be approximately so, for the flanges diminish in weight from the centre of the span to the supporting piers, but the web-bars increase in weight in the same direction; so, taking the weight as uniformly distributed, that coming upon each apex will be $= 16{\cdot}6 \div 9 = 1{\cdot}844$ tons, therefore the total load at each apex will be $10{\cdot}844 + 1{\cdot}844 = 12{\cdot}688$ tons.

In this case the loads fall on the bottom flange connections, but the parallelograms of stress will have the same proportions as those shown in Fig. 34, and described in Chapter I., in which the relations between apex loads and direct stresses are dealt with, and the following expressions determined (page 57).

Let W = the apex load; L = length of web-bars; D = depth of girder; and B = base of one triangle; all dimensions being measured between the intersections of centre lines.

Then,

$$\text{the thrust (or pull) on web-bar} = W \times \frac{L}{D}$$

$$\text{thrust on top (or pull on bottom) flange} = W \times \frac{B}{2\,D}$$

The character of the stress will depend upon whether the load comes upon the top or bottom flange first; in the present example it is on the bottom flange.

W in these expressions represents the part of the apex load which passes as stress along any particular bar, and this I must make clear before proceeding to the summation of stresses. If we take the load at 17 for instance, the part of it which goes to pier C' is $= W \times \frac{4}{10}$; and that which goes to pier D is $= W \times \frac{6}{10}$.

In each direction the stress due to the proportion of load passes equal in intensity through each bar to the piers, but the stress is alternately tensile and compressive; if therefore, we take each apex load separately, some tensile and other compressive stresses will be found on each bar, and their differences will be the resultant stresses on the bars. If the girder is fully loaded throughout its length, then all web-bars inclining upwards from the centre apex 15 towards A and B will be in tension, and those inclining downwards towards C and D will be in compression; but if the bridge is not fully loaded this will not occur, for the kinds of stress will be determined similarly, but on each side of the centre of gravity of the load on the bridge instead of from the centre of span. We have here, then, two sets of stresses to deal with, one equally distributed from the weight of the structure itself, and the other varying with the running load. The stresses upon the flanges are at a maximum when the bridge is fully loaded, which simplifies the calculations bearing upon them; I shall deal with these stresses first and determine them by the principle of moments. The stress on any flange segment will act about the connection opposite its centre and in the other flange—the moment which puts stress upon, for example, $3 - 4$, acts about the point 14.

At each of the points 12 to 20, there is concentrated a load of 12·688 tons, acting vertically downwards; therefore, there is an upward reaction at 11 equal to the load coming upon that point which is $= 12{\cdot}688 \times 4{\cdot}5 = 57{\cdot}096$ tons. I will show one working in detail and then proceed to summarize the flange stresses.

I will determine the stress upon the segment 4 — 5 of the top flange; the moments act about the point 15. The reaction acts at a distance of 4 bays $= 9 \times 36$ feet from the point 15, and it acts upwards; the downward moments are from the apex loads at 12, 13, and 14, and act at the distances $9 \times 3 = 27$ feet; $9 \times 2 = 18$ feet; and 9 feet from point 15. The resultant moment of stress will therefore be,

$$M = 57{\cdot}096 \times 36 - \{12{\cdot}688 \times (27 + 18 + 9)\} = 1370 \text{ foot tons},$$

if $S =$ the total resistance of the flange area.

$$M = S \times d = S \times 7\,794,$$

and, therefore,

$$S \times 7{\cdot}794 = 1370 \text{ feet tons},$$

whence,

$$S = \frac{1370}{7{\cdot}794} = 175{\cdot}77 \text{ tons}.$$

Taking the top flange first, the stresses in compression will be—

On segment 1—2—$S = 57{\cdot}096 \times 9 \div 7{\cdot}794 = 65{\cdot}94$ tons.

" 2—3—$S = \{57{\cdot}096 \times 18 - 12{\cdot}688 \times 9\} \div 7{\cdot}794 = 117{\cdot}21$ tons.

" 3—4—$S = \{57{\cdot}096 \times 27 - (12{\cdot}688 \times [18 + 9])\} \div 7{\cdot}794 = 155{\cdot}10$ tons.

" 4—5—$S = \{57{\cdot}096 \times 36 - (12{\cdot}688 \times [27 + 18 + 9])\} \div 7{\cdot}794 = 174{\cdot}40$ tons.

" 5—6—$S = \{57{\cdot}096 \times 45 - (12{\cdot}688 \times [36 + 27 + 18 + 9])\} \div 7{\cdot}794 = 186{\cdot}38$ tons.

The tensile stresses upon the bottom flange will be found by taking the moments about the points 1, 2, 3, 4, and 5. The stresses being symmetrical there is no necessity for taking them for more than half the length of the girder.

On segment 11—12 — $S = 57 \cdot 096 \times 4 \cdot 5 \div 7 \cdot 794 = 32 \cdot 96$ tons.

" 12—13 — $S = \{57 \cdot 096 \times 13 \cdot 5 - 12 \cdot 688 \times 4 \cdot 5\} \div 7 \cdot 794 = 91 \cdot 75$ tons.

" 13—14 — $S = \{57 \cdot 096 \times 22 \cdot 5 - (12 \cdot 688 \times [13 \cdot 5 + 4 \cdot 5])\} \div 7 \cdot 794 = 135 \cdot 54$ tons.

" 14—15 — $S = \{57 \cdot 096 \times 31 \cdot 5 - (12 \cdot 688 \times [22 \cdot 5 + 13 \cdot 5 + 4 \cdot 5])\} \div 7 \cdot 794 = 164 \cdot 83$ tons.

" 15—16 — $S = \{57 \cdot 096 \times 40 \cdot 5 - (12 \cdot 688 \times [31 \cdot 5 + 22 \cdot 5 + 13 \cdot 5 + 4 \cdot 5])\} \div 7 \cdot 794 = 179 \cdot 48$ tons.

The stresses upon the web-bars must now be determined, but a maximum moving load does not give the maximum stresses on all the bars at once, a moving load brings the maximum stress due to that load upon each web-bar in turn. This must be taken by itself and added to the stress due to the permanent load.

The load at any apex will be divided between the points of support in proportions relative to its distance from each. The load at 16 will be divided equally between them, as it is in the centre, and the stresses upon the web-bars will be alternately tensile and compressive. A load at 15 will be divided differently, $\frac{4}{10}$ will come upon D, and $\frac{6}{10}$ upon C; the part going towards D will put tensile stress upon 15 — 5, which received compressive stress from the load at 16; thus we get two series of stresses, and the resultant stress on any bar is their difference.

It is not, however, necessary to work out the stresses for each apex load separately for each web-bar. The relation of stress to load is as the length of the web-bar to the depth

of the girder, and this, for a web formed of equilateral triangles, is 1·154 to 1 (see page 58).

The running load for each cross-girder connection with the main girders, is 13·3 tons, and for each main girder $13 \cdot 3 \div 2 = 6 \cdot 65$ tons; and the stress on tie or strut accruing from this load, will be $6 \cdot 65 \times 1 \cdot 154 = 7 \cdot 674$ tons. By reducing the load to stresses in the first place, considerable labour in calculation is saved.

When the moving load covers the bridge, the stresses on bars 12 – 1, and 1 — 11, are those due to $4\frac{1}{2}$ apex loads; the bar stress $= 7 \cdot 674 \times 4 \cdot 5 = 34 \cdot 533$ tons, tension on 12 – 1, and compression upon 11 — 1.

Let the load now pass off the apex 12. There will be eight loaded apices, and the centre of gravity of the loads will be midway between the points 16 and 17. The 8 stresses $= 7 \cdot 674 \times 8 = 61 \cdot 392$ tons; the distance of the centre of gravity of the load from point 21 is $= 9 \times 4 \cdot 5 = 40 \cdot 5$ feet; so the stresses passing through the bars 13 — 2, and 2 — 12, will be $= (61 \cdot 392 \times 40 \cdot 5) \div 90 = 27 \cdot 625$ tons. This stress will also pass through bars 12 — 1 and 1 — 11, but as they are not maximum stresses on those bars, are not to be noticed, and generally the stresses thus running through bars to the left of those immediately dealt with, do not concern our calculations. Let the load now pass clear of point 13, then the seven loads left will have their centre of gravity at the apex 17; 36 feet from 21, and the maximum stresses on 14 — 3 and 3 — 13 will be $= (7 \cdot 674 \times 7 \times 36) \div 90 = 21 \cdot 487$ tons. Moving the load another bay there are six apex loads having their centre of gravity midway between points 17 and 18, therefore distant 31·5 feet from 21; and the stresses upon the bars 15 — 4 and 4 — 14 will be $= (7 \cdot 674 \times 6 \times 31 \cdot 5) \div 90 = 16 \cdot 103$ tons. After the next move there will be five apex loads, having

their centre of gravity at 18; 27 feet from 21. So the stresses on $16 - 5$ and $5 - 15$ will be $= (7{\cdot}674 \times 5 \times 27) \div 90 = 11{\cdot}502$ tons. When the load has reached the next bay, there will be four apex loads with their centre of gravity midway between 18 and 19, and at a distance of 22·5 feet from 21; so the stresses upon $17 - 6$ and $6 - 16$ will be $- (7{\cdot}674 \times 4 \times 22{\cdot}5) \div 20 = 7{\cdot}668$ tons. In the next position there will be three apex loads with their centre of gravity at apex 19; 18 feet from 21; the stresses upon $18 - 7$ and $7 - 17$ will be $= (7{\cdot}674 \times 3 \times 18) \div 90 = 4{\cdot}43$ tons. Next there are two loads, with their centre of gravity midway between 19 and 20; 13·5 feet from 21, the stresses upon $19 - 8$ and $8 - 18$ will be $= (7{\cdot}674 \times 2 \times 13{\cdot}5) \div 90 = 2{\cdot}3$ tons.

There is no necessity to carry these calculations farther as we have already passed the point at which compressive stresses, due to non-uniform loads, practically make struts of bars which, under a uniformly distributed load, would be ties. The structural provision to be made in these cases is in form of section as well as in sectional area.

The permanent load, due to the weight of the structure, ballast, and permanent way, will cause symmetrical stresses of the web-bars, so only one half of the web need be dealt with, and this can be taken as carrying half the load.

The permanent load will be equal to the total load, less the running load per apex, that is to $12{\cdot}688 - 6{\cdot}65 = 6{\cdot}038$ tons. On the bars $16 - 5$ and $5 - 15$ there will be a stress due to half this load, and at 15, 14, 13, and 12, twice this stress will be added. The stress from the half load at 16 will be $= (6{\cdot}038 \times 1{\cdot}154) \div 2 = 3{\cdot}484$ tons.

The stresses due to the permanent load will therefore sum up as follows :—

On bars	16—5 and 5—15,	the stress	= 3·484 tons.
,,	15—4 and 4—14,	,,	= 3·484 + 2 × 3·484 = 10·452 tons.
,,	14—3 and 3—13,	,,	= 10·452 + 2 × 3·484 = 17·420 ,,
,,	13—2 and 2—12,	,,	= 17·420 + 2 × 3·484 = 24·388 ,,
,,	12—1 and 1—11,	,,	= 24·388 + 2 × 3·484 = 31·356 ,,

Under the uniformly distributed permanent load only all bars inclining upwards towards the points A and B are in tension, and those inclining downwards towards C and D are in compression, but the stresses caused by the moving load do not follow this law; therefore the two series of stresses must be combined to give the resultant maximum stresses on each bar.

The following summation will give the maximum stress on each bar. + signifies compression, and − tensile stress.

On bar	1—11	stress	= + 34·5 + 31·4 =	+ 65 9 tons.
,,	12— 1	,,	= − 34·5 − 31·4 =	− 65·9 ,,
,,	2—12	,,	= + 27·6 + 24·4 =	+ 55·0 ,,
,,	13— 2	,,	= − 27·6 − 24·4 =	− 55·0 ,,
,,	3—13	,,	= + 21·5 + 17·4 =	+ 38·9 ,,
,,	14— 3	,,	= − 21·5 − 17·4 =	− 38·9 ,,
,,	4—14	,,	= + 16·1 + 10·5 =	+ 26·6 ,,
,,	15— 4	,,	= − 16·1 − 10·5 =	− 26·6 ,,
,,	5—15	,,	= + 11·5 + 3·5 =	+ 15·0 ,,
,,	16— 5	,,	= − 11·5 − 3·5 =	− 15·0 ,,
,,	6—16	,,	= + 7·7 − 3·5 =	+ 4·2 ,,
,,	17— 6	,,	= − 7·7 + 3·5 =	− 4·2 ,,
,,	7—17	,,	= + 4·4 − 10·5 =	− 6·5 ,,
,,	18— 7	,,	= − 2·3 + 10·5 =	+ 8·2 ,,
,,	8—18	,,	= + 2·3 − 17·4 =	− 15·3 ,,

The stresses on the central bars change from + to − with the movement of the load, thus bars 5—16 and 16—6 must be prepared to withstand 15 tons tension, and 4·2 tons compression, which means that it must not be a

flat bar, but made of angle or tee-iron. The resistance of a bar 7·794 feet long and ½-inch thick to compressive stress would only be under 0·9 ton per sectional square inch, and $4·2 \div 0·9 = 4·66$ square inches. The tensile stress only calls for $15 \div 5 = 3$ square inches, which will also be sufficient if angle-irons are used.

The sizes of the pins connecting the web-bars with the flanges must next be determined, taking the working shearing stress at 4 tons per sectional square inch. They will, of course, be the same for both halves of the girder. If D = diameter in inches and S = stress in tons; $S = 4D^2 \times 0·7854$; $D = \sqrt{S \div 3·141}$ for pins in single shear, and $D = \sqrt{S \div 6·282}$ for double shear, which occurs in the present example. This may be written $= \sqrt{S \div 2·5} = 0·4 \sqrt{S}$.

Taking the stresses from the foregoing table, the diameters of the pins will be—

At points	12, 1, and 11,	diam. of pin	$=0·4 \sqrt{65·9}=3·24$	ins. say	$3\frac{1}{4}$ ins.
,,	13 and 2	,, ,,	$=0·4 \sqrt{55·0}=2·96$	,,	3 ,,
,,	14 and 3	,, ,,	$=0·4 \sqrt{38·9}=2·49$	,,	$2\frac{1}{2}$,,
,,	15 and 4	,, ,,	$=0·4 \sqrt{26·6}=2·06$	,,	$2\frac{1}{8}$,,
,,	16 and 5	,, ,,	$=0·4 \sqrt{15·0}=1·55$	,,	$1\frac{5}{8}$,,

Where rivets are used, the sum of their sectional areas must equal the sectional area of the pin they displace.

The gross sectional areas for the top flange will be found by treating each segment as a strut.

The width of the flanges has been fixed at 2 feet, the depth will be 9 inches of the general section shown in Fig. 66, the side plates being joined to the top ones by angle-irons 3 inches × 3 inches × ½-inch thick. The least width of the segment will then be 9 inches; its length is 9 feet, therefore keeping 4·5 as the factor of safety, the working

stress to resist crippling will be thus found. The ratio of length to diameter or width is $9 \times 12 \div 9 = 12$, then

$$S = \frac{19}{1 + \frac{(12)^2}{900}} \times \frac{1}{4{\cdot}5} = 3{\cdot}71 \text{ tons per sectional sq. inch.}$$

Sectional area of segment	1—2 =	65·94 + 3·71	= 17·74	sq. inches.
,, ,, ,,	2—3 =	117·21 + 3·71	= 31·53	,,
,, ,, ,,	3—4 =	155·10 + 3·71	= 41·72	,,
,, ,, ,,	4—5 =	174·40 + 3·71	= 46·91	,,
,, ,, ,,	5—6 =	186·38 + 3·71	= 50·10	,,

Fig. 66 will suit the first segment, its sectiona l area is somewhat larger than is necessary, but it is as light as it is convenient to make it. The area amounts to — angle-irons $= 2\,(3'' + 3'' - \frac{1}{2}'')\,\frac{1}{2}'' = 5{\cdot}5$ square inches; top plate 24 inches $\times \frac{3}{8}$ inch $= 9$ square inches; side plates $2\,(9 \times \frac{1}{2})$ $= 9{\cdot}9$ inches; total $5{\cdot}5 \times 9 \times 9 = 23{\cdot}5$ square inches. Segment 2—3 requires 9·03 square inches more, which can be made up by adding a plate 24 inches $\times \frac{3}{8}$-inch thick on the top, as shown in Fig. 67. Segment 3—4 requires a further addition of 10·19 square inches to be supplied by another plate 24 inches $\times \frac{7}{16}$, which gives a total sectional area of 42·5 square inches. Segment 4 — 5 wants 4·41 square inches more, which a 24 inch $\times \frac{1}{4}$ plate will give, and the central segment 5—6 needs another 2·6 square inches, which is got by making the top plate $\frac{3}{8}$-inch thick instead of $\frac{1}{4}$-inch.

The bearing stress of the pins against the inside of holes in which they rest should not exceed 4·2 tons per square inch; the bearing area is equal to the diameter of the pin multiplied by the thickness of the metal through which it passes. The bearing areas required will be, at point 1, $= 65{\cdot}9 \div 4{\cdot}2 = 15{\cdot}7$ square inches. To make up this area

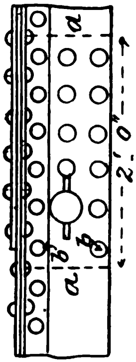


Fig. 68.

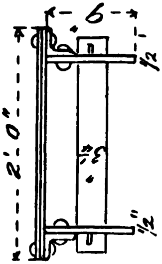


Fig. 67.

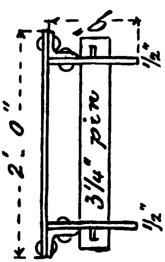


Fig. 66.

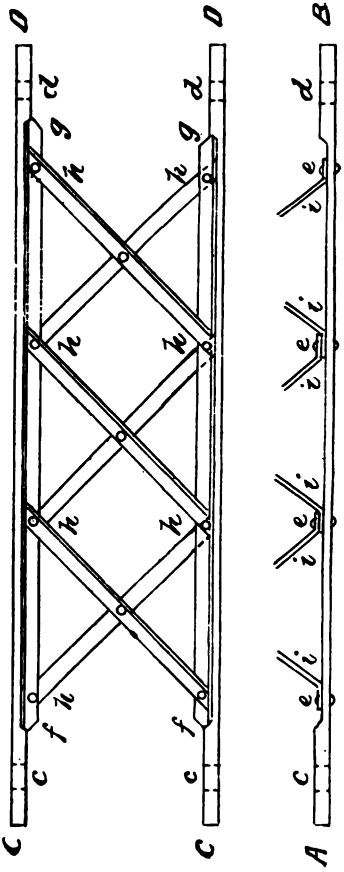


Fig. 69.

the total thickness of side-plates must be $15 \cdot 7 \div 3 \cdot 25$ (diameter of pin) $= 4 \cdot 6$ inches. So for a short length thickening plates must be riveted inside the side-plates, as shown by dotted lines in Fig. 68, which shows joint 2 in Fig. 62. At the end connection 1, the thickness of the side-plates must be increased on each side to 2·3 inches, so three ½-inch and one ⅜-inch plate will be necessary. The lengths of these bars will be in proportion to the bearing each will give. The bearing on the ⅜-inch plate will be $3 \cdot 25 \times 0 \cdot 375 = 1 \cdot 218$ square inches. If ¾-inch rivets are used, the shearing area for each will be 0·44 square inch; therefore the number of rivets in this plate must be $1 \cdot 218 \div 0 \cdot 44 = 3$. The rivets will be in three rows in each side-plate, so one row will do for the inside plate. Each of the other plates will give a bearing $= 3 \cdot 25 \times 0 \cdot 5 = 1 \cdot 625$ square inches, and require additional rivets to the number of $1 \cdot 625 \div 0 \cdot 44 = 4$ rivets, so the total number of rivets to connect these plates with the side plates of the flange will be fifteen, that is, five vertical rows, which, at a pitch of 4 inches, will make the plate next to the side plate 1 foot 8 inches long. At joint 2 the bearing area required is $55 \div 4 \cdot 2 = 13 \cdot 1$ square inches, and total thickness therefore $= 13 \cdot 1 \div 3$ (diameter of pin) $= 4 \cdot 3\dot{3}$ inches. Three ½-inch plates and one ¼ on each side will do this. The ¼ plate has a bearing area of $3 \times 0 \cdot 25 = 0 \cdot 75$ square inches, and will require $0 \cdot 75 \div 0 \cdot 44 = 2$ rivets, and each ½-inch plate has a bearing $= 3 \times 0 \cdot 5 = 1 \cdot 5$ square inches, and requires an additional $1 \cdot 5 \div 0 \cdot 44 = 4$ rivets, so the total number will be fourteen rivets, which may be arranged, as shown in Fig. 68, with two extra rivets, *b b*, to keep the plates close.

The thickening plates at the other joints will be dealt with in the same way; it is not necessary to occupy space here by taking each in detail.

The section of the bottom flange will be similar to that of the top, but inverted; the sectional areas of the segments of the bottom flange will be, taking the working stress at 5 tons per sectional square inch, as follows :—

Sectional area of segment	11—12 =	32·96 ÷ 5 =	6·59	sq. inches.
,, ,, ,,	12—13 =	91·75 ÷ 5 =	18·35	,,
,, ,, ,,	13—14 =	135·54 ÷ 5 =	27·11	,,
,, ,, ,,	14—15 =	164·83 ÷ 5 =	32·96	,,
,, ,, ,,	15—16 =	179·48 ÷ 5 =	35·89	,,

These are nett areas, the rivet and pin-holes being deducted from the gross area as the stress is tensile. The end section, if kept of the same sizes as those given in Fig. 66, will be, bottom plate $(24 - 4 \times 0{\cdot}75)\,\frac{5}{16} = 6{\cdot}5$ square inches; side-plates $2\,(9 - \{2 \times 0{\cdot}75 + 3{\cdot}25\})\,0{\cdot}5 = 4{\cdot}25$ square inches; and angle-irons $2\,(3 + 3 - \{0{\cdot}5 + 2 \times 0{\cdot}75\})\,0{\cdot}5 = 4$ square inches; making the total sectional area $= 6{\cdot}5 + 4{\cdot}25 + 4 = 14{\cdot}75$ square inches. Segment 12—13 will require 3·6 square inches more, which will be added in a plate 24 inches × ¼ inch. This brings the nett sectional area to $14{\cdot}75 + 6 = 20{\cdot}75$ square inches. Segment 13—14 requires 27·11 square inches, an increase of 6·36 square inches. A plate $\frac{5}{16}$-inch thick will give this. Segment 14—15 requires 32·96 square inches, therefore an addition of a $\frac{5}{16}$-inch plate, which brings the effective area up to $27{\cdot}89 + (21 + \frac{5}{16}) = 34{\cdot}45$ square inches. Segment 15—16 must have an effective sectional area of 35·89 square inches. Another plate ¼-inch thick will supply this. The plates should not in wrought iron be made more than 21 feet long, and they are more convenient to handle if they can be kept shorter. The lengths of the cover-plates will be determined by methods already described on pages 105 and 154; but it may here be pointed out that the joint-

covers on the vertical side-plates can be utilised also as stiffening plates to give bearing for the joint-pins.

In girders where the side-plates of the flanges are sufficiently wide apart, the struts may most conveniently be made of two bars each, as shown at *A A*, Fig. 64, these being laterally stiffened by bracing-bars, *i i*, &c. The least width of the combination is to be taken in determining the working resistance of the material. The bars which have no compressive stresses will not require stiffening.

I will show the application of the strut formula to one pair of bars, 1—11. The maximum stress is 65·9 tons. Let the width of the bar be 9 inches; then, as its length is 9 feet, $r = 12$, and the working resistance with 4·5 as factor of safety is

$$S = 19 \div \left\{1 + \frac{(12)^2}{900}\right\} \times \frac{1}{4{\cdot}5} = 3{\cdot}71 \text{ tons per sectional square inch.}$$

The sectional area required will be $65{\cdot}9 \div 3{\cdot}71 = 17{\cdot}7$ square inches. If each bar is made one inch thicker, there will be 18 square inches of gross sectional area to resist the compressive stress. In Fig. 69 is shown the edge of one bar with the ends thickened to give the required bearing on the pins at *c* and *d*. The bracing bars, *i i*, &c., may be made 4 inches × ½ inch thick, and fastened to the strut-bars by rivets, *e e*, &c. At *C D* is shown another method of connecting the bracing-bars. An angle-iron *f g* is riveted to each of the strut-bars, and angle-iron braces, *h h*, &c., are riveted to the return limbs of the angle-irons; this makes a very stiff strut. The remaining struts will all be calculated in a similar manner.

In making the tie-bars the ends may also be thickened. This is done by forging or pressing, not only to give bear-

ing, but also to make up for the loss of metal by the pin-holes.

Passing from this example to generalities, it is obvious that the value $\frac{L}{d}$ will vary with different angles of web-bars, and this must be determined for each particular case. If there are more than one series of triangles, as in Figs. 35 and 36, the load is not to be divided by 2, but the stresses on the web-bars of each series should be calculated separately; the stresses upon the flanges will be found by the principle of moments in the usual way.

If the load comes upon the top flanges of the bridge in the first instance, the mode of procedure is the same, but each load will commence by putting compressive stress upon the first pair of bars supporting it.

In girders of less depth a good form of flange consists of horizontal plates with deep angle-irons, between which the ends of the struts and ties are held. The struts in such cases may conveniently be made of tee-iron or steel, riveted together back to back with a plate between them if necessary, and for very light girders angle-irons or steels may be used, placed back to back where they cross, for both struts and ties. In those cases where the series of triangulation are so numerous as to form a lattice web, the flanges can be calculated in the same way as those of a plate-girder.

When the ties and struts cross each other it is usual to rivet or bolt them together at their intersections, which helps to give lateral steadiness, but this must not be taken as shortening the effective lengths of the struts in regard to their capacity for resisting compressive stress.

In cases concerning bridges of long span, the connections between the web-members and the flanges will often be made more conveniently by means of rivets, than the

e pins which would become necessary, as these would ssitate very heavy thickening plates for bearing, which ts will not require. The great objection to these t-plates in small girders is that they disfigure it, but he larger structures they are not so conspicuous.

n this matter—as in so many others—no general rule be laid down, but the designer must decide, according he circumstances of each case, what form of struts and nections will be most suitable for his purpose.

CHAPTER X.

TRUSSED BRIDGES.

TAKEN in a broad sense all triangular and lattice web-girders are included under the term trussed girders, for they may all be regarded as built up of a combination of trusses, but the term here is used in reference to a distinct type, of which two examples are shown in Figs. 70 and 71.

In the triangular and in the lattice web-girders there is a top and bottom flange, but in the trussed girders here shown there is no bottom flange; the floor of the bridge is necessarily carried upon the top of the main girders.

Although one or two attempts have been made to introduce this type in England, it has never found favour with English engineers. There is not the rigidity which is found in girders with top and bottom flanges. I shall therefore deal only with the general mode of determining the stresses, but not enter into the details of design in reference to sectional areas and connections which can be settled in the same way as those previously considered.

In Fig. 70 *A B* is one of the main girders carrying a bridge floor, and it is trussed by tie rods running from various points in its length to the lower ends of vertical struts *c k*, *d l*, *e m*, &c. The same formula used previously for converting load into stress will also apply in this case; the ratio of stress on any tie-bar to the load taken up by it

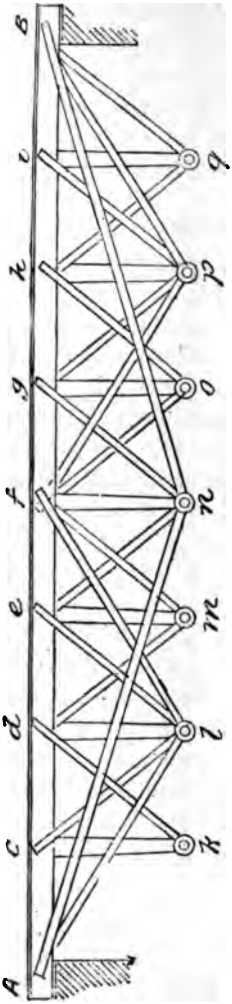


Fig. 70.

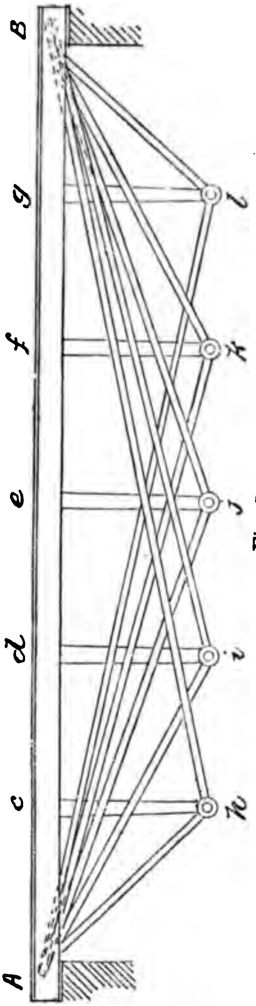


Fig. 71.

will be that of its length to the length of the strut from which it takes its load. In the example the girder is divided into eight bays by the struts, so on each strut there will be one-eighth of the total load on the girder; let W = this load; L = the length of any tie; and D = the length of the struts, all these lengths being taken from centre to centre.

The load upon $c\ k$ is taken up by the ties $k\ A$, $k\ d$, half on each; one half or $W \div 2$ becomes vertical load upon A, for although the stress upon $k\ A$ is more than this, the stress is resolved horizontally and vertically at A, and this will occur with all the loads. It will be seen shortly why I am treating the loads in this way. The other half of the load on c goes to d, which also has half the load from e in addition to its own load, making 2 W altogether: this stress passes down $d\ l$, and half the load is taken by tie $l\ A$, and the other half by tie $l\ f$ to f, that is W to each. From e there also passes $W \div 2$ through $m\ f$ to f. There will therefore be at f W from $d\ l$; $W \div 2$ from $e\ m$; similar loads on the other side from $h\ p$ and $g\ o$ and its own load W; this makes a total of $1 \cdot 5\ W \times 2 + W = 4\ W$, of which half goes to each point of support; therefore 2 W comes upon A, and the total load brought there by ties $k\ A$, $l\ A$, and $n\ A$ will be $= W \div 2 \times W + 2\ W = 3 \cdot 5\ W$. As there are seven struts, and the loads are uniformly distributed, there would necessarily be half seven strut-load on each support, which shows that $3 \cdot 5\ W$ is correct, and therefore the loads taken up by the ties are also accurately determined.

Some additional steadiness may be obtained by connecting the bottoms $k\ l\ m\ n\ o\ p\ q$ together by light tie-bars carried by the same pins which at these points connect the tie-bars with the struts.

Let the effective span of the girder be 40 feet, and the effective strut length 6 feet $= D$; then from the properties of right-angled triangles the lengths of the ties can be determined. The length of each bay will be 5 feet, as the girder length is divided into eight parts by the seven struts. The length of the ties kA kd md mf of oh qh and qB will be

$$L = \sqrt{5^2 + 6^2} = 7{\cdot}81 \text{ feet.}$$

Let the load per lineal foot be 1·2 tons, then the load $W = 1{\cdot}2 \times 5 = 6$ tons, and the stress upon bars kA kd md mf and symmetrically placed tie-bars in the other half of the girder will be—

$$S = \frac{\text{W}}{2} \times \frac{L}{D} = \frac{6 \times 7{\cdot}81}{2 \times 6} = 3{\cdot}905 \text{ tons.}$$

For the ties lA lf pf and pB, the length will be—

$$L = \sqrt{\overline{10}^2 + 6^2} = 11{\cdot}66,$$

and the stress upon them—

$$S = \frac{2\,\text{W}}{2} \times \frac{L}{D} = 6 \times \frac{11{\cdot}66}{6} = 11{\cdot}66 \text{ tons.}$$

The length of the ties nA nB will be—

$$L = \sqrt{\overline{20}^2 + 6^2} = 20{\cdot}88,$$

and the stress upon them will be—

$$S = \frac{4\,W}{2} \times \frac{L}{D} = 12 \times \frac{20{\cdot}88}{6} = 41{\cdot}76 \text{ tons.}$$

The compressive stress upon the top member—which is *constant throughout its length*—is found, by resolving the

stresses on the ties vertically upon the support and horizontally upon $A\ B$ and totalling the results. The horizontal compression will be to the tension as the horizontal length is to the length of the tie. The horizontal lengths will be 5 feet, 10 feet, and 20 feet;

$$\text{The thrust from tie } k\,A = 3{\cdot}905 \times \frac{5}{7{\cdot}81} = 2{\cdot}5 \text{ tons}$$

$$\text{,, ,, ,, } l\,A = 11{\cdot}66 \times \frac{10}{11{\cdot}66} = 10{\cdot}0 \text{ ,,}$$

$$\text{,, ,, ,, } n\,A = 41{\cdot}76 \times \frac{20}{20{\cdot}88} = 40{\cdot}0 \text{ ,,}$$

$$\overline{52{\cdot}5} \text{ ,,}$$

The mode of trussing shown in Fig. 71 is different. Each load is carried directly to the points of support by the ties connected to the bottoms of the struts. Let the span be 40 feet as before; it is divided into six bays of $6{\cdot}6\dot{6}$ feet each, the depth D will also be taken as 6 feet as in the previous example. A lead of 1·2 tons per lineal foot gives $1{\cdot}2 \times 6{\cdot}6\dot{6} = 8$ tons on each strut. By determining first the proportion of load taken off by a tie the same formula for the stresses can be used as above.

The lengths of the ties will be—

$$h\,A \text{ and } l\,B - \sqrt{\overline{6{\cdot}6\dot{6}}^2 + 6^2} = 8{\cdot}97$$

$$i\,A \text{ and } k\,B - \sqrt{\overline{13{\cdot}3\dot{3}}^2 + 6^2} = 14{\cdot}61$$

$$j\,A \text{ and } j\,B - \sqrt{\overline{19{\cdot}99}^2 + 6^2} = 20{\cdot}87$$

$$l\,A \text{ and } h\,B - \sqrt{\overline{33{\cdot}33}^2 + 6^2} = 33{\cdot}85$$

$$k\,A \text{ and } i\,B - \sqrt{\overline{26{\cdot}6\dot{6}}^2 + 6^2} = 27{\cdot}31$$

Of the load at c, five-sixths goes to A, and one-sixth to B; of that at d four-sixths goes to A and two-sixths to B; and of that at e half goes to each point of support; the stresses upon the ties will therefore be—

$$\text{On } h\,A \text{ and } l\,B - S = \mathrm{W} \times \frac{5}{6} \times \frac{L}{D} = \frac{8 \times 5 \times 8{\cdot}97}{6 \times 6} = 9{\cdot}96 \text{ tons.}$$

$$\text{,, } i\,A \text{ and } k\,B - S = \mathrm{W} \times \frac{4}{6} \times \frac{L}{D} = \frac{8 \times 4 \times 14{\cdot}61}{6 \times 6} = 12{\cdot}99 \text{ ,,}$$

$$\text{,, } j\,A \text{ and } j\,B - S = \mathrm{W} \times \frac{1}{2} \times \frac{L}{D} = \frac{8 \times 1 \times 20{\cdot}87}{2 \times 6} = 13{\cdot}92 \text{ ,,}$$

$$\text{,, } l\,A \text{ and } h\,B - S = \mathrm{W} \times \frac{1}{6} \times \frac{L}{D} = \frac{8 \times 1 \times 33{\cdot}85}{6 \times 6} = 7{\cdot}52 \text{ ,,}$$

$$\text{,, } k\,A \text{ and } i\,B - S = \mathrm{W} \times \frac{2}{6} \times \frac{L}{D} = \frac{8 \times 2 \times 27{\cdot}31}{6 \times 6} = 12{\cdot}14 \text{ ,,}$$

The horizontal thrusts upon the member $A\ B$ will be—

$$\text{The thrust from tie } h\,A = 9{\cdot}96 \times \frac{6{\cdot}6\dot{6}}{8{\cdot}97} = 7{\cdot}47 \text{ tons.}$$

$$\text{,, ,, ,, } i\,A = 12{\cdot}99 \times \frac{6{\cdot}6\dot{6} \times 2}{14{\cdot}61} = 11{\cdot}85 \text{ ,,}$$

$$\text{,, ,, ,, } j\,A = 13{\cdot}95 \times \frac{6{\cdot}6\dot{6} \times 3}{20{\cdot}87} = 13{\cdot}34 \text{ ,,}$$

$$\text{,, ,, ,, } l\,A = 7{\cdot}52 = \frac{6{\cdot}6\dot{6} \times 5}{33{\cdot}85} = 7{\cdot}45 \text{ ,,}$$

$$\text{,, ,, ,, } k\,A = 12{\cdot}14 \times \frac{6{\cdot}6\dot{6} \times 4}{27{\cdot}31} = 11{\cdot}85 \text{ ,,}$$

$$51{\cdot}96 \text{ ,,}$$

The final horizontal stresses in the two systems are nearly equal.

CHAPTER XI.

CANTILEVER BRIDGES.

ALTHOUGH the adoption of the cantilever form of bridge is comparatively recent in the United Kingdom, yet it is an ancient mode of construction. The advantages of this system were advocated by Mr. Sedley, who proposed by this means to bridge the Thames about the same place which is now occupied by the Tower Bridge. Plans were prepared and models made in 1861-2, but the matter went no farther then, and it seemed to drop out of sight until revived again in connection with the "Sukkar" Bridge in India and the "Forth" Bridge in Scotland.

In regard to continuous girders and those with fixe ends, it has been shown that each span is in principle com posed of two or three parts; if a girder is carried over tw spans, each span will be in effect part cantilever and par beam, the cantilever part being over and on each side o the central pier, from the end of each cantilever part th rest of the girder acts as a girder with ends freely sup ported. If the girder is continuous over three spans, th centre span will be as two cantilevers carrying a freel supported girder between them. The cantilever ends an the girder begins at the point of "contra-flexure"; th position of this point, however, varies with alterations o

load, so that, in the continuous girder, neither the girder nor the cantilever is permanently defined as to length or span. The idea of cutting the flanges at a convenient point of contraflexure occurred, and the cantilever bridge construction thus reached. One form is shown, in elevation, in Fig. 72. It consists of three spans, *A B*, *B E*, and *E F*, the loads on the end spans will counterbalance that on the centre span, or if they are insufficient for this, the ends at *A* and *F* must be anchored down to substantial foundations. The part *C D* is simply a girder-bridge with free ends supported by the ends of the cantilevers, *B′ B C* and *E′ E D*. The stresses upon the members *A B′*, *B′ C*, and *D E′*, *E′ F* will be tensile, and those on the horizontal members, *A B C* and *D E F* will be compressive. The condition of *C D* is that of an ordinary freely supported bridge and needs no comment here; it may be carried on bed-plates arranged to allow for contraction and expansion in a way which will be explained in a subsequent chapter. Let the spans measured from the faces of the abutments to the centres of the piers be—centre span 1,000 feet, end spans 500 feet each, bridge *C D* 200 feet, leaving for *B C* and *D E* 400 feet each.

To fully consider all the details of the design of such a bridge as this would occupy at least all the space at my disposal, I must, therefore, deal only with the general conditions of stress.

Taking the dead weight of the structure as being practically uniform, the balance upon the piers *B* and *E* will be about right, but when a moving load occupies the centre span, this equilibrium will be disturbed, and therefore it will be necessary to anchor the ends of the cantilevers at *A* and *F*.

If the bridge carries a single line of railway, there will

be a maximum moving load of 1·5 tons per lineal the main girders; therefore 0·75 tons per lineal each girder; this would be a test load made up es to fill the whole span, but for this provision sho made.

The moving load at C and D will be $0{\cdot}75 \times 100$ tons; this, taken in regard to its moment about the of the pier E, will be $75 \times 400 = 30{,}000$ foot tons. moving load upon DE will be $0{\cdot}75 \times 400 = 300$ its centre of gravity will be 200 feet from E, and the its moment about that point will be $300 \times 200 = 6$ foot tons, making a total moment of 90,000 foot tons. divided by the length of the cantilever EF, gives 90,0 $500 = 180$ tons, anchorage to be provided on each ment. By increasing the length of the end cantilevers weight of anchorage may be reduced, but whether thi be done or not will depend upon the conditions that d mine the spans.

The stresses upon the top and bottom members o cantilevers will be determined by the principle of mon and those upon the web-bars by the parallelogra forces.

In bridges consisting of more than three spans w not get the absolute fixture of the anchorages at the ments in the intermediate spans; the piers must, ther afford a base sufficiently wide to prevent any lifting bed-plates under the pier-struts BB' and EE'. The struts over the piers are shown by gi and hk; the four of these on each pier to each girder, each set firmly braced together, so as to form a rigid tower. condition of stability in this case is that the sum c live and dead loads on one span acting about the foc the strut hk, shall not give a greater moment of w

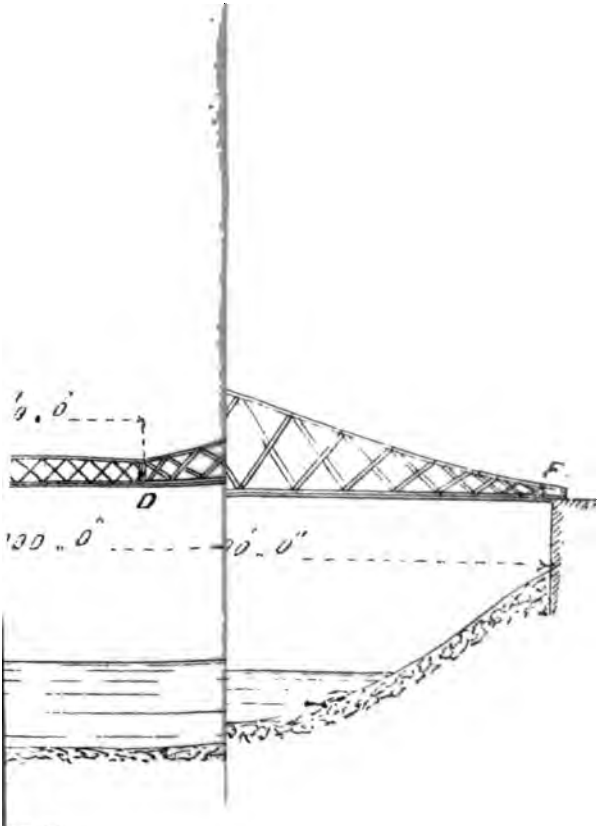


Fig. 72.

[To face p. 194.

than that accruing about the same point (h) from the dead load only in the next span; otherwise the foot of the strut gi would be lifted from its base plate.

It is obvious that the cantilever principle is not suitable for bridges of small span in which the weight of the structure itself is small compared with that of the moving load.

CHAPTER XII.

BRIDGES SUPPORTED BY ARCHES.

BRIDGES carried by arches may be divided into two clas and the arches which carry them may also be divided i two classes.

The roadway or platform of the bridge may be of same construction as that for any other kind of bridge; the first division is—first, bridges with the roadway ab the arch; second, bridges with the roadway below the a The second division has reference to the arch itself; it i be supported by abutments designed to take the thrust the arch may be tied; that is to say, the thrust on the a is taken up by a tie, running from haunch to haunch, then vertical load only comes upon the piers.

In Chapter II. we found by means of a diagram (Fig. that under a uniform load an arch would be stable if rise or versine did not exceed a fifth of the span, the a being circular in form. So, keeping within that ratio, calculations for an arch may be made by means of formulæ determined in that chapter.

Let $T =$ the thrust at the crown of the arch; $T' =$ thi at any point distant x feet from the crown of the ar $l =$ effective span in feet; and $v =$ rise or versine in f I shall take as an example a public road bridge, shown elevation in Fig. 73, and in transverse section in Fig.

A B C is the arch, upon the top of which is carried a spandril-girder *D E*, and to this are connected the cross-girders of the bridge floor. The spandril-girder is supported upon the back of the arch by upright struts, *f f*, &c., which are braced in the centres of their lengths by a tie *g*. The roadway is carried by two main arches, which are braced together at intervals by tee-bars, *h h*, &c., to secure lateral rigidity, so that the arch need not be treated as a column.

The effective span of the arch is 100 feet $= l$; its rise, 15 feet $= v$; and the width of roadway 25 feet. The material to be used is steel, having compressive resistance equal to 35 tons and tensile 30 tons; so with 5 as the factor of safety, the working stresses will be:—In compression, $35 \div 5 = 7$ tons per sectional square inch of gross area; and in tension $30 \div 5 = 6$ tons per sectional square inch of nett area.

The bridge floor may be made on any of the systems previously described. Here I shall deal with the main ribs and their bracing only.

In regard to load, if the bridge is crowded with people as close as they can be packed, the live load will not exceed 120 lbs. per square foot of flooring, and vehicular traffic will not amount to as much. The road metalling and floor I will assume at 140 lbs. per square foot; this will make the total load upon the two arches $= \{120 + 140\} \times$ 100 feet span $\times$ 25 feet width $=$ 650,000 lbs. $=$ 290 tons. The rise of the arch approximates one-seventh of the span, and the weight factor for this (see Table, page 82) is 0·00108. The weight, then, of each rib, inclusive of pandrils, will be

$$145 \text{ tons} \times 0{\cdot}00108 \times 100 \text{ feet span} = 15{\cdot}66 \text{ tons},$$

and the total load to be carried will be 14
= 160·66 tons; $w = 1·606$ tons.

On these figures the thrust at the crown o
will be

$$T = \frac{w\,l^2}{8\,v} = \frac{1·606 \times \overline{100}^2}{8 \times 15} = 133·8\dot{3} \text{ tons}$$

The sectional area required at the crown of the
therefore be $= 133·8\dot{3} \div 7 = 19·12$ square inc
section may be built up as shown at F, Fig. 73; t
9 inches × ⅜-inch, horizontally; one web-plate 24
⅜-inch; and four angle-steels 3 inches × 3 inches
The gross sectional area thus obtained will be

2 plates 9″ × ⅜″ . . =	6·75	square	inches.
1 plate 24″ × ⅜″ . . =	9·00	,,	,,
4 . L . S . 3″ × 3″ × ⅜″ =	8·43	,,	,,
	24·18	,,	,,

showing a margin of 5 square inches. We sh
however, use smaller sizes than these for such a s

The thrust at the haunch will be

$$T' = \sqrt{\left(\frac{w\,.\,l^2}{8\,v}\right)^2 + (w\,x_1)^2} =$$

$$\sqrt{(133·83)^2 + (1·606 \times 50)^2} = 156 \text{ tons}$$

which will require a gross sectional area of
= 22·28 square inches; so the section already
will be sufficient all through the length of the a
web-plates are to be stiffened every 4 feet with
× 3 inches × ⅜-inch tee-steels riveted on both s
packing pieces under them between the main ang

Now as to covers for th· · · ·. Wherever jo

D
B
A

in the web-plates the tee-steels on each side will serve as covers, but the best job would be obtained by making the arch in pieces, say, 16 feet long, and planing the ends truly to fit, having at these ends angle-steels on both sides in place of tee-steels. The joints would then be made by bolting or riveting the angle-steels together, as shown in horizontal section at G, Fig. 73. The web-plate can be made in shorter lengths to save waste in following the curve and its joints made by the tee-steels, which will occur between the end angle-steels. If this accurate fitting is not obtainable, then the flange-plates and angle-steels must be connected by cover-plates in the usual way.

The struts f may be put at distances of 5 feet apart; then each must be proportioned to carry a maximum load of $5 \times 1{\cdot}606 = 8{\cdot}03$ tons. The length to be taken for determining the working resistance will be that of the longest one, as it will be convenient to keep them all of one section; this will be 45 feet from the crown of the arch, where the upper surface of the arch is in contact with the under surface of the spandril-girder $D\,E$, which is horizontal. The radius of a circular arc is equal to the square of the chord divided by eight times the versine plus half the versine. In this case, then,

$$R = \frac{l^2}{8\,v} + \frac{v}{2} = \frac{\overline{100}^2}{8 \times 15} + \frac{15}{2} = 90{\cdot}8\dot{3} \text{ feet.}$$

If $O =$ an ordinate distant x_1 from the crown of the arch, and drawn at right angles to the tangent under surface of the spandril-girder, this will be the length of the spandril strut which occurs at x_1 feet from the centre of the span. As the arch is 2 feet deep the radius of its extrados will be $90{\cdot}83 + 1 = 91{\cdot}83$ feet, and the length of the strut 45 feet from the centre will be.

$$O = R - \sqrt{R^2 - x_1^2} = 91{\cdot}83 - \sqrt{(91{\cdot}83)^2 - (45)^2}$$
$$= 11{\cdot}78 \text{ feet.}$$

The struts may be made of tee-steels 6 inches × 3 inches, riveted together back to back; the least width will be that measured diagonally, shown by y on the section H, Fig. 73. This will be the diagonal of a square whose sides are 3 inches, and therefore $= \sqrt{2 \times 3^2} = 4{\cdot}242$ inches. The ratio of length to width will be 11·78 feet × 12 ÷ 4·242 = 33·3. Taking 5 as the factor of safety, the working stress per sectional square inch will be

$$S = 19 \div \left\{1 + \frac{r^2}{900}\right\} \times \frac{1}{5} = 19 \div \left\{1 + \frac{(33{\cdot}3)^2}{900}\right\} \times \frac{1}{5}$$
$$= 1{\cdot}7 \text{ tons.}$$

The sectional area of the strut must therefore be not less than 8·03 ÷ 1·7 = 4·72 square inches. Two tee-steels 6 inches × 3 inches × $\frac{3}{8}$ inch will give a sectional area of 6·469 square inches, which shows an extra margin of strength.

In this case the spandril-girder is required as a means of connecting the bridge floor with the arch, the actual stress upon it will be very slight. If the cross-girders of the bridge floor join the spandril-girder just over the struts, the loads will come directly upon the latter, and if they join it midway, the stress will not be much; we will take it in this way, and also disregard the continuity of the spandril-girder; then we shall have a series of girders 5 feet in span with a central load of 8·03 tons. For purposes of connection, the spandril-girder should be 1 foot deep, then the maximum stress on either flange at the centre will be

$$S = \frac{Wl}{4d} = \frac{8{\cdot}03 \times 5}{4 \times 1} = 10{\cdot}037 \text{ tons.}$$

quiring a sectional area in compression = 10·037 ÷ 7 1·43 square inches, and in tension 10·037 ÷ 6 = 1·67 uare inches. A web ¼ inch thick, with two angle-steels, inches × 3 inches × ⅜ inch as top and bottom flanges, ill have ample strength.

I will now take another example, this time of a tied arch idge, carrying the bridge floor below the arch. It is own in elevation in Fig. 75, and in transverse section in ig. 76. The span and rise are the same as in the previous ample, the material the same, but the bridge is assumed be designed to carry two lines of railway, the cross-rders will be 10 feet apart, and carried at the lower ends suspension members, *e e*, &c. The main arch *A B C* is ed at its haunches by the main tie *A C*.

The liability of the arch to distortion is resisted by iagonal counter-bracing, *f f*, &c. Applying the principle of oments to this construction, it is evident that the stress pon the main tie is constant throughout its length, and qual in intensity to the thrust at the crown of the arch.

There being two main girders, and two lines of railway, ere will be one line to be carried by each main rib or rch; from the table of loads we find the uniformly dis-ributed load for this span is 145 tons per line of railway, nd as there are nine suspension members, which divide he bridge into ten bays, this will bring 14·5 tons on each ispension member, when the bridge is fully loaded; this ill be taken for the stress on the arch or main tie, but ach suspension rod is liable to a greater local load if two comotives are standing side by side with their heaviest riving wheels on one cross-girder, this will bring a live oad of 16 tons on the suspension member; there is also in

each case the weight of the structure to be added; this I shall take, including weight of ballast, permanent way, and flooring, at 0·8 tons per lineal foot.

The live load per lineal foot is 1·45, and the weight of floor, ballast, &c., 0·8 tons, making $1·45 + 0·8 = 2·25$ tons, and therefore the total load, from external sources will be $2·25 \times 100 = 225$ tons. To determine the weight of the tied arch, we may use the factors for plate-girders; they will be somewhat in excess, but a reduction of area can, if necessary, be made afterwards if economy is to be strictly studied. The table does not go higher than $\frac{1}{8}$ of the depth, but as the weight of the running section varies inversely as the depth, $\frac{1}{8}$ can be taken off the factor, for 8 to give that for the 7 which will be $0·00182 - 0·00022 = 0·0001\dot{6}$. Then the weight of the tied arch itself will be,

$$= 225 \text{ tons} \times 0·00182 \times 100 \text{ feet span} = 40·95 \text{ tons},$$

the total weight carried by each rib $= 225 + 40·95 = 265·95$; and $w = 2·66$ nearly.

The thrust at the crown of the arch, and the tensile stress throughout the tie will be,

$$T = S = \frac{w \,.\, l^2}{8\,v} = \frac{2·66 \times \overline{100}^2}{8 \times 15} = 221·6\dot{6} \text{ tons.}$$

The thrust at the haunch will be,

$$T' = \sqrt{(221·6\dot{6})^2 + (2·6\dot{6} \times 50)^2} = 258·67 \text{ tons.}$$

The sectional area required at the crown of the arch will be $221·6\dot{6} + 7 = 31·66$ square inches, and that at the haunches $= 258·67 \div 7 = 36·97$ square inches. If we make the top and bottom plates at the crown of the arch, 12 inches $\times \frac{1}{2}$ inch, the vertical or web-plate 24 inches $\times \frac{3}{8}$, and the connecting angle-steels 3 inches $\times$ 3 inches

16' 0"
of rail

× ½ inch, the gross sectional area will be 32 square inches. At the haunches we want nearly 37 square inches, 5 inches more; by making the web-plate ⅝ inch thick, we add 6 inches, this will do for the haunch sections, but as the stress increases directly, the crown is the exact section, and not practically sufficient there, so from the ¼ span on one side to that on the other side of the crown, the web may be made ½ inch thick.

The nett sectional area required in the tie is $221{\cdot}6\dot{6} \div 6 = 33{\cdot}61$ square inches. In order to find the loss by rivet-holes, we must find how many rivets will be required in shearing stress to attach the cross-girder end to the main tie of the arch.

The permanent load, as the cross-girders are 10 feet apart, will be $0{\cdot}8 \times 10 = 8$ tons, and the maximum live load on one end will be 16 tons, 24 tons in all. Taking the rivets as having ⅘ the tensile resistance, the working resistance will be $6 \times 0{\cdot}8 = 4{\cdot}8$ tons per sectional square inch. If ⅞-inch rivets are used, the area of each cross-section is 0·6 square inches, and its shearing resistance $0{\cdot}6 \times 4{\cdot}8 = 2{\cdot}88$ tons. The number of rivet areas required will be $24 \div 2{\cdot}88 = 8{\cdot}3$, that is 9 rivets. There would be used 10 rivets, 5 to each of the end angle-steels of the cross-girder; bringing these in between the rivets, fastening the main angle-steels to the web of the main tie. The vertical rivet-pitch will be about $3{\cdot}6\dot{6}$ inches, and there will be 7 rivet-holes to deduct from the width of the web-plate. The main tie will be kept to the same outside dimensions as the arch; the top and bottom flanges will have two rivet-holes each to be deducted, and the angle-steels one each, as these latter, being zig-zag, cannot both come in line with the vertical row of rivets in the web. For ⅞-inch rivets, I should use a 5-inch pitch; and the angle-steels would be

4 inches $\times$ 4 inches $\times$ $\frac{1}{2}$ inch thick. The four will give as nett sectional area $4 \{3\cdot5 - 0\cdot875\} \frac{1}{2} = 13\cdot25$ square inches. A $\frac{1}{2}$-inch web-plate with 7 rivet-holes out gives $(24 - 6\cdot125) \frac{1}{2} = 8\cdot937$ square inches, and two $\frac{5}{8}$-inch flange-plates, with two rivet-holes out of each give $(12 - 1\cdot75) \frac{5}{8} = 12\cdot81$ square inches. This makes up a total nett area of 34·997 square inches.

The cross-girders throughout the structure are connected to the main tie immediately below the suspension members, so that in no case can they bring transverse stress upon the main tie. Particular attention must be given to the arrrangement of the connection between the arch and the tie at the ends, here the rivets must be sufficient to pick up the whole of the stress upon the haunch of the arch ; that is 258·67 tons. The number of $\frac{7}{8}$-inch rivets required for this will be $258\cdot67 \div 2\cdot88 = 89\cdot8$, that is 90 rivets.

It will be found convenient to have the end web-plates at each end in one piece, to serve both for the arch and tie, this plate will be of the form shown at *h h*, and its right-hand inclined edge being strengthened by two angle-steels, one each side will form an abutment for the arch to rest upon, the web of the arch also having angle-steels riveted on it, and riveted also to the other angle-bars as shown at *g g*. The bottom main angle-steels of the arch are bent up, and carried along horizontally, as shown at *i i*, where they are riveted through the top main angle-steels of the main tie. The bottom flange-plate of the arch, and the top flange-plate of the tie terminate where they meet at *i*, and are there connected together by a gusset-plate *k*, which is riveted through them, and their main angle-steels through the attachment of bent angle-steels on each side.

The web of the main tie is further connected to the plate *h h*, by cover-plates *m m* placed on both sides of the web

plates, so that all the rivets passing through these covers and the web-plates beneath, will be in double shear, so that each rivet represents the strength of two. An enlarged elevation of this joint is shown at Fig. 77.

The suspension members *e* are now to be considered; the load upon each is the maximum load at one end of a cross-girder, and this has been found to amount to 24 tons. The nett sectional area of each member *f* must therefore be not less than $24 \div 6 = 4$ square inches. As it is advisable to take every means to minify lateral vibration or oscillation, these members may with advantage be made of rigid form although their special duty is to resist tensile stress only. Ten rivets have been allowed for the connection of each end of a cross-girder with the main girder, and the same number must be used to connect the main tie with the suspension member, if they are exposed to shearing stress; if they are in tension fewer may be used, as the tensile working resistance is 6 tons, which will give each $\frac{7}{8}''$ rivet a carrying capacity of $0 \cdot 6 \times 6 = 3 \cdot 6$ tons; the number required would be $24 \div 3 \cdot 6 = 6 \cdot 6$, that is seven rivets. Details of this joint with the main tie are shown in elevation in Fig. 78, and in section in Fig. 79; the connection with the arch is similar but the cap of the suspension member will be shaped to fit the curve of the soffit of the arch.

I should make this member of four-angle steels *b b*, &c., each $4'' \times 4'' \times \frac{3}{8}''$ with their ends bent at right angles to their height as shown at *e e*, and between the outer and inner pairs a stiffening plate *g g* will be riveted to act as a double gusset to the two bends, the same being done also at the top of the member; the central parts of the inner and outer angle-steel will be joggled down to meet each other and be solidly riveted together along the length

between the stiffening or gusset-plates. There will be four rivets on each side of the foot.

The nett sectional area of one angle-steel will be $= \{4 + 4 - (0{\cdot}375 + 2 \times 0{\cdot}875)\}\,\frac{3}{8} = 2{\cdot}203$ square inches and of the four together 8·812 square inches, more than twice that necessary, but in cases of this kind the economy of one member must be sacrificed for the convenience of the general requirements of the work. The vertical lines of rivets *ff*, are those which connect the end of the cross-girder *D* with the web of the main tie *A*.

In order that the lines of rivets in the foot of the suspension member should naturally fall opposite those in the tie angle-steels, the gusset-plate should be the same thickness as the web-plate of the tie.

The gusset-plates *g* will also afford a means of connecting the bracing-bars *f* with the arch and tie. We cannot determine any definite stress upon these, and are guided by what we have found satisfactory in practice; these may be made 8 inches wide and $\frac{3}{8}$-inch thick and riveted together when they cross in the centre.

In order to keep the tops of the arches steady, bracing frames *H H* should be riveted to them, there being in this case five of these, 5 feet apart over the centre of the span; they must be arched up when necessary to allow for the required headway, which should not be less than 15 feet above rail level.

These cross-braces will be of light material, say two angle-steels $3'' \times 3'' \times \frac{3}{8}''$ with $\frac{3}{4}''$ rivets for top and bottom flanges, depth 18 inches, as the width is 25 feet, and the flanges connected by lattice bars $3'' \times \frac{3}{8}''$ at 45 degrees to the horizon. The feet to connect these bracing frames with the top flange of the arch must be made $4'' \times 4'' \times \frac{1}{2}''$

to take the 5″ pitch of the rivets, and each foot will be connected with the arch by eight rivets. An end plate is riveted between the 3″ × 3″ angle-steels at each end of the frame.

If the floor of the bridge is not braced by buckled or other continuous plates, horizontal bracing must be fixed diagonally in each bay between the cross-girders. This may be made of 4-inch by 6-inch × ½-inch tee-steels, back to back, riveted together where they cross, and with their webs bent at the ends and riveted together through the cross-girder webs, as shown at *c* in Fig. 79. These ends may, if necessary, be riveted to the web or bottom flange of the main tie, but no extra rivets must be used for this purpose, as the loss of section caused thereby has not been allowed for in calculating the sectional area of the tie; and it is always advisable to avoid putting more rivets than are absolutely necessary in members under tensile stress.

The ordinary cover-plates along the arch and tie will be arranged in the usual way.

CHAPTER XIII.

CHAIN BRIDGES.

In Figs. 80 and 81 are shown an elevation and a central transverse section of a chain bridge—the usually accepted form of suspension bridge. The bridge shown consists of three spans, the centre being 100 feet, and the side ones 50 feet each, to the centres of the piers and the faces of the abutments.

The bridge is carried by main chains *A B C D E*, passing over the tops of the towers *B F* and *D G*, and having their ends at *A* and *E* carried down into the ground and anchored behind masonry or concrete as shown in detail at Fig. 82.

The roadway, which may be of any of the types previously described, is carried by continuous lattice-girders *K K K*, having effective spans of 10 feet, and attached at every ten feet to suspension rods *h*, the upper ends of which are supported by the main chains. The girders *K*, in order to give rigidity to the floor, should be calculated for a span of 30 feet so as to distribute partial loads over four suspension rods. These girders will be calculated according to the formulæ exemplified in Chapter IX, but a close lattice, which also serves as a parapet to the roadway, is most suitable. By inclining the chains inward, towards the centre, as shown in the cross-section,

Fig. 81, the lateral stability of the roadway is improved, as explained in Chapter II; the suspension rods being also inclined in the same plane as the chains. The height of the girders K should bring them 4 feet above the level of footway in a road bridge, which is the class I shall take for this example.

The material used will be wrought iron, with a working tensile resistance of 5 tons per sectional square inch; and, in shearing, 4 tons per sectional square inch.

The versine or dip of the chain in the centre span will be taken as one-sixth, $100 \div 6 = 16 \cdot 6$ feet.

The live load upon the roadway at 120 lbs. per foot super., and the weight of ballast, asphalt and floor-girders, including girders K, I will take at 160 lbs. per square foot, making the total load to be carried by the two chains equal to 280 lbs. per square foot; the bridge is 26 feet wide, there fore the load per lineal foot of span upon the two chains will be $280 \times 26 = 7{,}280$ lbs $= 3 \cdot 25$ tons; that is $1 \cdot 625$ tons per lineal foot upon each chain.

The same formulæ used for determining the stresses upon the arch apply also to the chain; therefore, the tension at the centre of the middle span will be,

$$T = \frac{w \,.\, l^2}{8\,v} = \frac{1 \cdot 625 \times \overline{100}^2}{8 \times 16 \cdot \dot{6}} = 120 \text{ tons}$$

dropping the decimals. The tension on the main chains, at the towers, will be

$$T' = \sqrt{\overline{120}^2 + (1 \cdot 625 \times 50)^2} = 145 \text{ tons nearly.}$$

The nett sectional areas of the chains will therefore require to be; at the centre of the span, $120 \div 5 = 24$ square inches; and at the towers $145 \div 5 = 29$ square

inches. Between these points, the sectional areas will vary by so small a quantity that it will be convenient to use the larger sectional area throughout the chain. Of course, in a chain-bridge of longer span, say 400 or more feet, it would be desirable to vary the sectional areas of the links, as the saving would be considerable.

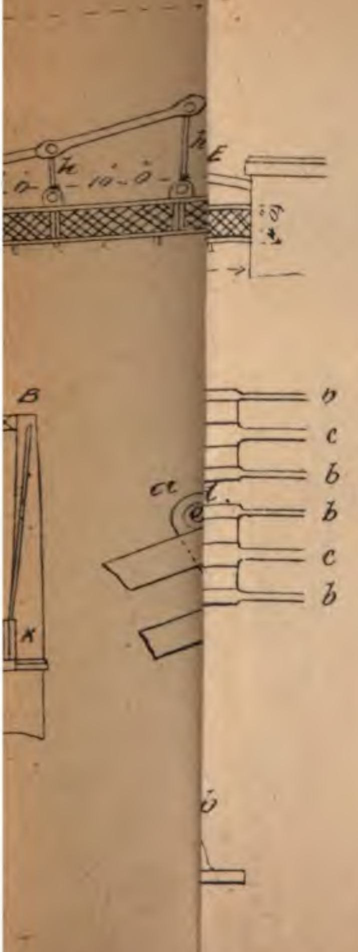
Something must be said here about the form and manufacture of these links, of which one is shown in elevation at Fig. 83. These should not be made of equal width throughout, but with eyes, O O, at each end, the body P, of the bar being proportioned to the stress which it will have to bear. Then comes the question of proportion of the eyes to the body of the bar. This is a point upon which we cannot theorize with any great satisfaction, so must base our sizes upon experiment and experience, which give the following results :—

The bar is assumed to be of the same thickness throughout ; it may fail by bursting open along the line q, or tearing asunder through one or both the lines r r. To get the full strength of the body in the eye, the length q should be made equal to $1\frac{1}{2}$ diameters of the pin-hole, and the two lines r r should be together equal to $1\frac{1}{4}$ times the width of the body of the bar.

These dimensions being set out, easy curves are used to form the outline of the eye. The main pins of the chains have to be arranged to take the load from the suspension rods, and this will be most conveniently done by means of joint plates, one on each side of the head of each suspension rod, which will place its connecting-pin in double shear.

The remarks made in reference to the proportions of the eyes in the main chains will also apply to eyes in other parts of the structure.

The arrangement of the main chain joints is a matter



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which requires very careful consideration, and this is the next matter to be dealt with.

The sectional area of chain to be supplied is 29 square inches; let the bodies of the links be $7\frac{1}{2}$ inches deep, then if the sum of their thickness be 4 inches, the chain will have a sectional area of 30 square inches.

In Fig. 84 is shown a plan of one joint and connection with the head *d* of a suspension rod. *e e* is the centre line or axis of the pin holding the parts together. It is seen that the bars *a* and *c* bring double shear upon the pin and the bars *b* bring single shear upon it, therefore it is obvious that the bars *b* should be half the thickness of bars *a* and *c*, the two latter will be made 1 inch, and the former $\frac{1}{2}$-inch thick. The stress brought upon a single section of the pin from the main chain will not exceed $7{\cdot}5 \times 0{\cdot}5 \times 5$ tons $= 18{\cdot}75$ tons. The stress brought upon the pin by the suspension bar is $1{\cdot}625$ tons $\times$ 10 feet $= 16{\cdot}25$ tons carried in double shear. The shearing area necessary would be $18{\cdot}75 \div 4$ tons $= 4{\cdot}687$ square inches, for which a pin $2\frac{1}{2}$ inches in diameter would give sufficient sectional area, but we also require the same amount of bearing area for each section in shear, so it will be better to make the pins $4\frac{5}{8}$ inches in diameter and thicken out the heads of the links, making those of *a* and *c* 2 inches, and those of *b* 1 inch thick.

The sectional area required for the suspension rod will be $= 16{\cdot}25$ tons $\div 5 = 3{\cdot}25$ square inches, which is satisfied by a rod $2\frac{1}{8}$ inch diameter, and keeping the head the same thickness there will be ample bearing upon the pin. The anchored ends of the chains (Fig. 82) are to be carried sufficiently far back into the ground to give a certain foundation, and the factor of safety for stability should not be less than 2. The ends of these links are made with massive heads to give sufficient bearing surface upon the pull resist-

ing plates *M M*, behind which the heads are further secured by a cotter *n*. The plates *M M* are built in with strong masonry in cement *l l*, which is imbedded in ballast concrete to form a solid mass, the quantity of which will depend upon the stress and the locality; if, for instance, the ground is rock, there is a ready-made anchorage, but if it is gravel the anchorage must be artificially formed.

The towers, which may be of masonry, brick, or iron, carry near their summits strong wrought iron or cast steel saddles placed upon rollers to allow for the slight movement caused by passing loads, which slightly distort the chains, or, instead of saddles on rollers, rocking pieces may be used, but whichever form is adopted, the full sectional area of the chain must be kept throughout it.

The connection of the head of the suspension rod with the chain, will allow for longitudinal movement, and the lower connection, with the parapet girder, must be made to accommodate any lateral oscillation without wrenching the rod. The form of this joint is universal, and it is shown in front and side elevations in Fig 85. The lower end, *a*, of the suspension rod is made with an eye, which rests between the lugs, *c c*, of the connecting piece, and has a bolt or pin, *b b*, passing through all of them; the lower part of the connecting piece forms another eye, *d*, at right-angles to the eye in the suspension rod, and held by a bolt, *e e*, to lugs, *f f*, which are secured to the roadway girder. These lugs may be continuations of bars that pass through the top flange of the roadway girder into the web, and so form suspension plates for the girder; the continuity of the structure is then well maintained.

The suspension rods may be inclined downwards towards the centre of the span to aid the stability of the superstructure, but in that case the stresses upon them will be

increased. It will be seen that in these matters stability is sought in giving the structure such a form that its elements will be under initial directive stress, with only its dead weight before any oscillatory forces come into action.

If two chains, one above another, are used, the suspension rod may be conveniently attached to a triangular plate, *a b c*, Fig. 86; by this the stress is distributed between the chains *A* and *A′*; *e* is a pin passing through and fixed in the suspension plate, *a b c*, and it rests freely upon the upper chain, *A*; the corner, *b*, is supported by the main chain pin, *d*, which passes through it, and at the lowest angle *c*, is a hole, *f*, to receive a bolt connecting it with the head of the suspension rod; the plates may be made in pairs to take the suspension rod between them, otherwise it would require a forked head with two eyes to give even bearing. In the alternate joints the arrangement can be so far reversed, that the lower pin is fixed in the suspension plate and rests freely upon the lower chain, whilst one of the main chain pins passes through the hole in the upper part of the plate, thus the fixed connection of the suspension rods will be alternately with the upper and lower chains, whereby they will be equally steadied.

Another method consists in joining the suspension rod heads directly to the main chain, but alternately to each.

It has been proposed to use two chains thus, but stiffened by lattice bracing; but this system does not seem rational to me, as the lattice-work must hinder the chains from adapting their forms to the varying stresses.

The arch and chain may be combined for large spans with excellent results, as shown in Brunel's splendid bridge at Saltash, which has two spans of 450 feet each, carrying a railway suspended from an arch of elliptical tubular section tied by suspension chains on each side.

CHAPTER XIV.

OPENING BRIDGES.

It not unfrequently happens that a bridge is required in a locality where a permanently fixed structure across a river is not admissible on account of the navigation, therefore a form of bridge which can be moved clear of the waterway when required must in such a case be adopted.

There are three descriptions of bridge available; first, the rolling bridge, which can be run back on to land, or one span which can be run back under in a long bridge or viaduct; second, a bridge mounted on a turn-table, so that it may be turned a quarter round, it will then lie lengthways of the stream and leave room for the vessels to pass; third, the "bascule" bridge, which turns on a horizontal axis and so reaching a vertical position leaves a clear way for ships. There must be nothing loose about the roadway of a "bascule" bridge.

The most satisfactory of these three types is the turntable bridge, but it is not always that it will fit the conditions of the locality. One suitable for a narrow stream or a canal is shown in side elevation at Fig. 87. Fig. 88 is a plan of the "live" rollers and lower or "dead" ring of the turn-table. Figs. 89 and 90 are details of the rings and centre-pin of the turn-table.

The bridge is to carry a roadway, and is 18 feet wide; the

width of the canal is 78 feet and the headway insufficient to allow boats to pass under. The main girders $A\ B\ C$ are shown as built up of plates; lattice-girders may be used if required, but for such small spans the plate-girder is more economical.

When this bridge is swung round there is a clear opening of 30 feet on each side of the central pier, that which supports the turn-table.

Two sets of calculations must be made for the main girders, one when supported at the centre and both ends, and fully loaded; the other for the permanent load only with the ends A and C clear of the abutments. In the first case this is a continuous girder of two equal spans, and the maximum stress will occur over the centre pier, where it will be, on each flange,

$$S = \frac{w \cdot l^2}{8\,d}$$

In which w = the total live and dead load per lineal foot, and l = 39 feet, the effective length of each span. Let w_1 = the dead load per lineal foot, when the bridge is open the flange stress will be that on a cantilever 39 feet long, or

$$S' = \frac{w_1 \cdot l^2}{2\,d}$$

The dead load, including the weight of the structure, with ballast or road metal and paving, will be about 28 cwt. = 1·4 tons per lineal foot of span. The live load, taken as 120 lbs. per superficial foot, will be 18 feet × 120 lbs. = 2,160 lbs. = 0·964 tons. Therefore w_1 = 1·4 tons and w = 1·4 + 0·964 = 2·364 tons per lineal foot of span. The stresses at the centre B or D, on either flange, for the two main girders will be :—

The bridge shut and with a full load,

$$S = \frac{w \, . \, l^2}{8 \, d} = \frac{2{\cdot}364 \times \overline{39}^2}{8 \times d} = \cdot$$

The bridge open and supporting dead lo

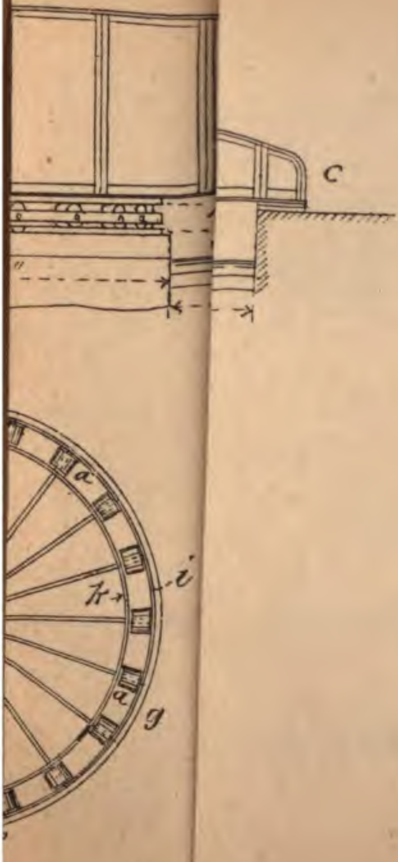
$$S' = \frac{w_1 \, . \, l^2}{2 \, d} = \frac{1{\cdot}4 \times \overline{39}^2}{2 \times d} = \frac{10}{}$$

If we keep the flanges of the same throughout their length, the girders may be in each direction from B to the bearings A will be seen that if proportioned to th when open, there will be ample strength Assume that the depth of girder practically c the abutments; the maximum stress betwee ment A and the first point of contraflexure is $\frac{3}{8}$ $= 14{\cdot}625$ feet; therefore, the girder depth at will be $d_1 = d \times \frac{14{\cdot}625}{39} = 0{\cdot}375 \, d$; and the f at this point with the bridge shut will be,

$$= \frac{w \, . \, l^2}{14 \, d_1} = \frac{2{\cdot}364 \times \overline{39}^2}{14 \times 0{\cdot}375 \, d} = \frac{684{\cdot}88}{d}$$

which is much less than $\frac{1064{\cdot}7}{d}$

The main girders may be advantageously m siderable depth over the central pier, say tapered down to 2 feet at each end. The st flanges at B and D, tension on the first and com the latter, will be $= 1064{\cdot}7 \div 8 = 133{\cdot}09$ ton material be steel with working resistances p square inch of 7 tons in tension, and 6 in c



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then the nett area at $B = 133{\cdot}09 \div 7 = 19$ square inches, and the gross sectional area at $D = 133{\cdot}09 \div 6 = 22{\cdot}18$ square inches, both of which can be obtained with the flange 12 inches wide, which is enough for the span. The further detailing will follow in the usual course.

The bearings at A and C are slightly inclined, for necessarily there is some deflection at the ends of the girder when the bridge is open, and this will gradually take it up as it closes and give the ends their due support.

I have not space to enter at length into the details of the turning gear, which is machinists' work, but the turn-table may be generally described. It is shown in plan in Fig. 88, in which a number of rollers, *a a*, &c., rest upon a bed-ring, *g g*; upon the tops of the rollers is carried a live ring, *h*, in section, Fig. 89, and in the elevation; upon this ring the main girders are bolted to a bed-plate which distributes the load over the ring. The rollers, as shown in the section, are truncated cones, the sides of which if produced would meet in the centre of the turn-table, and thus there is no rubbing friction between the rollers and the rings. These rollers are retained in position by rings *i* and *k*, carried by the axle-rods, *b b*, &c., which, at their inner ends, are fixed in a ring *c* (shown in section in Fig. 90), which revolves on a seating round a central pin *d* fixed in a cast steel bed-plate *m*, which is securely bolted down on the central pier. Upon the pier is fixed a toothed circular rack, of which part is shown at *e*, Fig. 88, and into this a toothed pinion, *e′*, gears; this pinion is fixed on the lower end of a shaft, *e f*, Fig. 87, the upper end of which is connected by gearing with winch handles upon the bridge whereby it may be rotated. In large bridges the turning is effected by means of hydraulic machinery.

If the conditions are such that the spans must be unequal, then the shorter span will be loaded to balance the longer; in this way a turning-bridge may be made which will consist of one span only, having its turn-table and loaded tail on one side of the stream.

If, however, the river banks are occupied by warehouses and wharfs, or other buildings, there will not be room to turn a bridge to the bank; the alternative is to use the rolling bridge.

The main girders of a bridge of this description must have the undersides of the bottom flanges planed to a true surface to run upon the bearing rollers over which they have to travel. The bridge must be continued inshore far enough to accommodate below its flooring a counter-balance to the overhanging dead weight when drawn back from the opposite shore, and this short end will be fitted with wheels which run on rails provided for the purpose. Now it is obvious that if a bridge is to be thus run back it must have some place to receive it. In the Parrett Bridge at Bridgewater, carrying a railway, a length of about 80 feet of the permanent way was carried upon a framing on which it was drawn away to one side on rails at right-angles to its length to leave a space into which the river span could be drawn, and after this was returned to its closed position, the permanent way on its carriage was replaced; the span of the bridge was 75 feet, and at its free end it was fitted with a parabolic beak to lift that end to its bearing; a steam winch with automatic gearing operated this bridge.

The friction of turn-table bridges properly constructed is not considerable; experience shows that it is usually greater in opening the bridge than in closing it, though that is not always the case.

Experiments upon eleven American bridges showed a maximum friction of 7·94 lbs. per 1,000 lbs. weight of moving structure; one bridge showed as low a friction as 3·53 lbs. per 1,000 lbs.

I have no records of friction of rolling bridges, but these should, if well-made and properly kept in order, certainly not exceed in friction the turn-table bridges, as they have fair bearings upon cylindrical rollers and straight rails.

The third form of opening bridge now requires attention, this is the "bascule," which divides in the centre and opens upwards in two leaves. The latest example of this is the Tower Bridge, London.

Figs. 91 and 92 are diagrams of a "bascule" bridge in elevation, the former shows it closed and the latter open.

The structure consists of two half-arches FB and GC, which, when closed, bear upon abutments HH, and meet at B and C, and so act as one arch, the stresses upon which may be determined in the usual way. There will be two arches, one on each side of the bridge, and they carry the roadway between them.

The roadway is continued in each direction, and is formed as tail pieces at each end to carry counterbalancing loads at A and D.

For the purposes of opening, each half of the structure is fixed on a massive axle or trunnion E, capable of carrying the whole dead weight when the half-arches are lifted from their abutments; this will put the trunnions E E under shearing stress, for the bearings must be kept close up to the arches, as any transverse stress would cause a jamming in the bearings of a very destructive character. In order to get a proper bearing surface the diameter of the trunnions will probably have to be larger

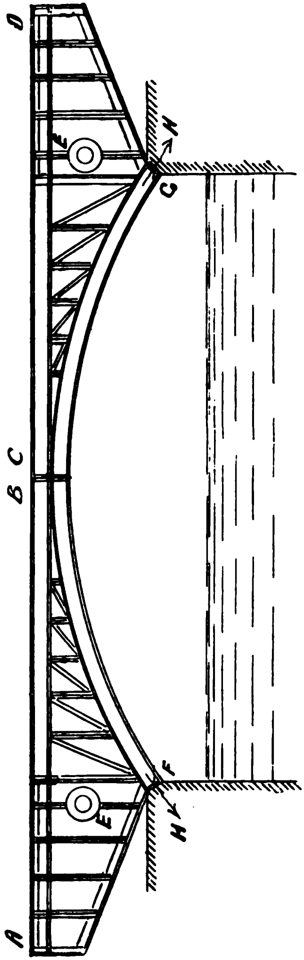

Fig. 91.

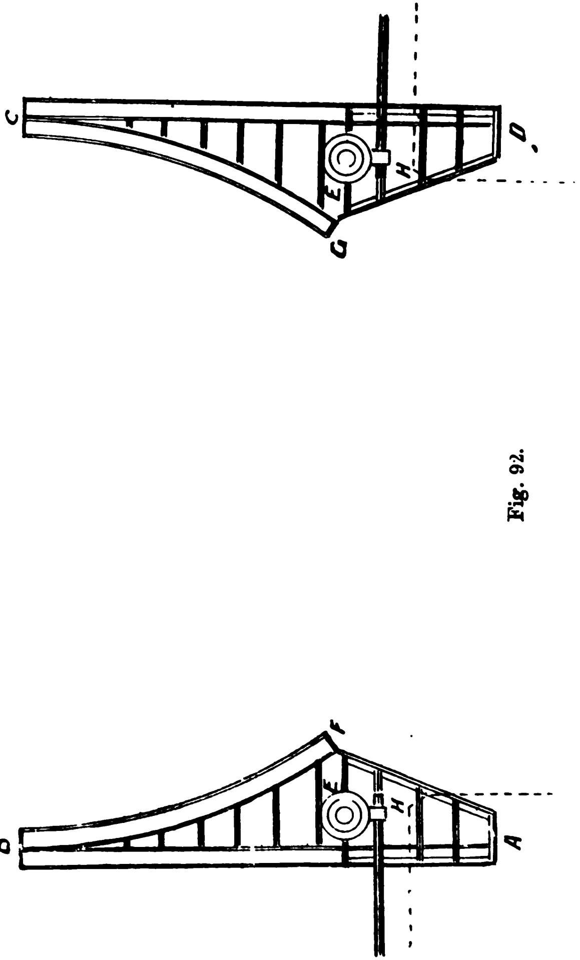

Fig. 92.

than that required for shearing stress. If the bridge is 100 feet span and 20 feet wide, the dead load will be about one and a half tons per lineal foot of span, which will give $1 \cdot 5 \times 50 = 75$ tons for each half-span. Then there is the counterbalance to be considered. If the tail-piece is 20 feet long the balancing load will be greater than the weight of the half-span in ratio to its shorter leverage. Assuming the centre of gravity of each load to be at the centre of the length, the counterbalance will weigh

$$75 \times \frac{25}{10} = 187 \cdot 5 \text{ tons.}$$

so that the total weight on each of the trunnions will be $187 \cdot 5 + 75 = 262 \cdot 5$ tons. If the trunnion is of iron, with a working resistance to shearing stress of 4 tons per sectional square inch, each shearing area must be,

$$\frac{262 \cdot 5}{2 \times 4} = 32 \cdot 8 \text{ square inches.}$$

This area would be given by a trunnion $6\frac{1}{2}$ inches in diameter. The bearing area must be equal to the shearing area, so if this diameter is adopted the length of bearing required would be $32 \cdot 8 \div 6 \cdot 5 = 5$ inches. For this length of bearing I should prefer a larger diameter to gain extra rigidity and obviate the wearing unduly of the bearings on the inner edges.

There are several ways in which these bridges may be opened and closed. If it is to be done by hand power, the trunnions E are made long enough to pass through the bearing blocks, and have worm-wheels fixed on their outer ends, or they may be carried farther, so as to have another bearing beyond the worm-wheel, and so obviate bending stress from the action of the worm. A worm-wheel is a

kind of toothed wheel, but the teeth are cut to receive a worm or screw fixed on a shaft *I*, by the revolution of which the worm-wheel is turned round; the worm-wheel is in fact a continuous section of part of a screwed nut. The great advantage of this combination is that the worm or "tangent screw" holds the worm-wheel in any position, and so acts as a lock to keep the bridge-leaf in any position required.

Another method consists in fitting the trunnions with toothed wheels in which racks gear, then by the direct movement of the racks the trunnions are rotated, and the racks may be conveniently operated by hydraulic machinery.

If the work is properly designed and constructed there should be no tendency to either drop or rise when unloaded, and practically there need not be, as no movement can be made without overcoming the friction of the trunnions in their bearings; but to prevent vibration, locking bolts will be fitted to steady the bridge-leaves when closed, and relieve the trunnions of the weight of the counterbalances, so that the arches carrying the bridge shall take their whole support from the abutments *H H*.

In some places lifting bridges, working in guides and raised by chains, have been constructed, but this method is not applicable to works of any magnitude; very strong towers would be necessary, and the chains would be cumbersome, and some difficulty might arise in keeping the bridge level in raising and lowering. In the turning, rolling, and "bascule" bridges the opening apparatus has merely to move the structure, not to support its weight as well.

CHAPTER XV.

IRON AND STEEL PIERS.

THE piers used for the supports of bridges and viaducts may be placed in three classes: masonry piers; iron cylinders or caissons filled with concrete or masonry, or a combination of both; and piers built up of groups of columns.

In the second class the iron serves the purpose of holding the materials together until they have set, but has no part in the subsequent support of the superstructure; the third class, grouped columns, is that with which I shall deal in this chapter, the load here rests directly upon the iron or steel used in the construction of the pier.

Fig. 93 shows an elevation of the top tier of columns in a pier of this class; a plan with the top girders removed is shown in Fig. 94.

It will be seen from these illustrations that the columns are so braced together, both vertically and horizontally, that the height of each, for the purposes of calculation, is that of the height of the tier to which it belongs; so that, by this arrangement, we may have a pier of any practicable height, without lowering the working resistance per sectional square inch of the columns.

In the example the pier is arranged to carry four girders *A A A A*, upon which the flooring of the bridge above is carried. This is a very convenient disposition of girders

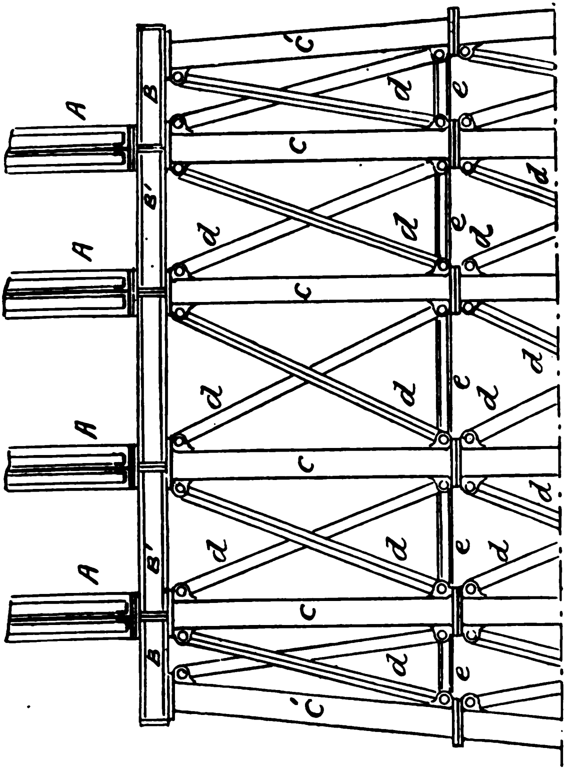

Fig. 93.

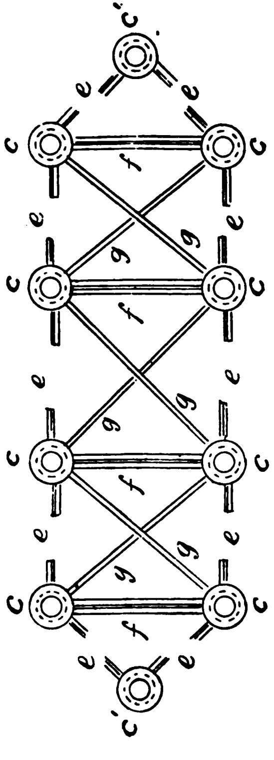

Fig. 94.

for a double line of railway, as a longitudinal main girder comes under each rail.

The tops of the columns are tied together and kept in position by girders $B\,B$ and $B'\,B'$, which also support the bed-plates upon which the main girders rest. These bed-plates and the bearing-plates will be dealt with in a subsequent chapter.

The columns $C\,C\,C\,C$ are the bearing columns, and $C'\,C'$ are "raking" columns to give lateral stability to the pier, though, for that matter, the columns C have a slight rake, to give the pier a broad base.

The columns are braced vertically—or nearly so—by the bars d, and horizontally by the bars $e\,f$ and g. All these bars, except those marked g, should be of rigid section. For the bars d, tee-irons or steels, placed back to back, are very suitable, and the bars e and f, which have to act mostly as struts, should be of I section. The steel joists, rolled for builders' work, will come in very well for this purpose.

For the piers of a viaduct of moderate height up to about 80 feet from the ground, or river-bed, as the case may be, 10 feet is convenient for the height of each tier of columns; a greater height reduces the efficiency of the bracing-bars d, as the horizontal distance between the central columns is only 6 feet, and that from these to the outer bearing columns 5 feet, measured, of course, from centre to centre of the columns.

For these structures then we at once get a working stress from the formula previously given—for cast iron, with 6 as a factor of safety, and 1 foot for diameter of column—

$$S = 36 \div \left\{1 + \frac{r^2}{400}\right\} \div 6 = 6 \div \left\{1 + \frac{100}{400}\right\}$$

$$= 4{\cdot}8 \text{ tons per sectional square inch.}$$

For structures of this kind it will be difficult to displace cast iron by wrought iron or steel tubes, for if these latter are used they must have cast metal caps and bases to supply lugs for the connection of bracing-bars; or else have a number of angle-pieces riveted to them for this purpose, and then it is exceedingly difficult to get enough rivets to take the stresses, and these always look clumsy and unreliable. Where no lug connections are necessary, an angle-steel ring will serve to connect, with a steel cap or base, all wrought; but although solid steel tubes can now be bought up to 20 inches diameter and $\frac{3}{4}$ inch thick, yet, unfortunately, no experiments upon their resistance to pressure as columns are available for deducing formula of practical utility.

Built-up columns of wrought steel may be treated by the formulæ used for angle, tee, and channel-steels, but such columns are not suitable to be used for high piers, though they are very useful for ground work, and the ribs by which the segments are connected serve for the attachment of such bracing bars as are required.

In one instance in my own experience rolled-iron joists have been used in place of columns for a railway viaduct in the neighbourhood of Halesowen. Four joists, 12 inches by 7 inches, were used in tier to carry a single line of railway, the height of the centre pier being about 100 feet.

Figs. 95, 96, and 97 show details of the connections of the bracings with the columns. In determining the sections of these bracings we are by necessity guided by experience only, for there are no data upon which to base calculations of stress.

For piers of this type I have always used $6'' \times 3'' \times \frac{1}{2}''$ tee-irons for the diagonal ties d, and generally the same for the horizontal struts e, until rolled joists were available,

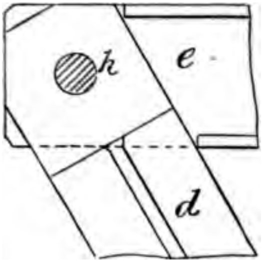

Fig. 95.

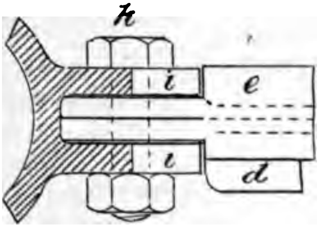

Fig. 96.

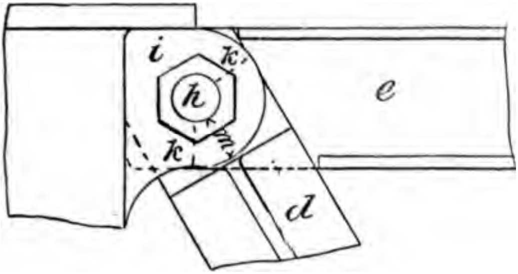

Fig. 97.

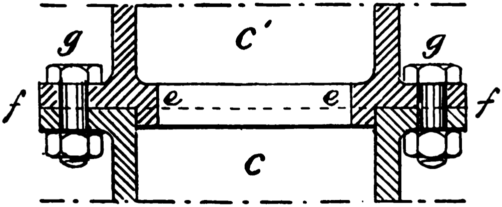

Fig. 98.

when $6'' \times 4'' \times \frac{1}{2}''$ bars became available. Now a $5'' \times 4\frac{1}{2}''$ steel joist would be more convenient, and is more rigid than a tee-iron.

Fig. 95 shows the ends of the struts *e* and the bracing-bars *d* in position upon the bolt *h*, which passes through lugs cast on the column, but not shown in this figure. The flanges of the strut *e* are cut away at the ends, as is also the web of the tee-iron *d*, to allow the ends of the vertical limbs of each to pass between the lugs *i i* of the columns, as shown in Figs. 96 and 97.

These ends will fit freely between the lugs, and the bolts be screwed up only tight enough to prevent their turning in the holes in the lugs. If rupture of the bracing occurs it will be most likely by breaking of the lugs through the lines *k k*, Fig. 97.

The bolt *h* should be made, in double shear, equal in strength to the end of the tee-iron *d*, with the web off. Its sectional area will be $6'' \times \frac{1}{2}'' = 3$ square inches, without deducting the bolt-hole; the working resistance of this area in tension will be $3 \times 5 = 15$ tons, and the sectional area of bolt required to sustain this will be $15 \div 4 = 3{\cdot}75$ square inches, therefore in each shearing section $3{\cdot}75 \div 2 = 1{\cdot}875$ square inches, which is given with some margin by $1\frac{5}{8}$-inch diameter.

The ultimate strength of cast iron in tension—such qualities as are generally used for bridge work—is about 7 tons per sectional square inch; so with 5 as a factor of safety, the working resistance will be $7 \div 5 = 1{\cdot}4$ tons per sectional square inch. Then the required area on the planes *k k* will be 15 tons $\times$ 1·4 tons $= 10{\cdot}71$ square inches. If we make the distance *m* from the bolt-hole to the outside of the lug $2\frac{1}{2}$ inches, the lugs will each have to be 2 inches thick.

To get the full strength of the tee-iron its table must be thickened up by welding a piece on to the end up to where the web is cut off.

The fitting together of the columns is a matter of great importance, for if these joints are not accurately made, there will be a serious loss of rigidity.

The best form of connection that can be used is shown in Fig. 98. The lower column C is bored accurately for a short distance, $1\frac{1}{2}$ inches is ample, at its upper end, to receive a turned spigot e e, at the base of the upper column C'. The faces of the flanges f are also turned or faced exactly at right angles to the axis of the columns, and in these flanges bolt-holes are drilled to templates to ensure exactitude of position, and the columns are bolted together with turned bolts g. Such a joint is equal to solid metal.

The bases of the bottom tiers of columns will be formed to suit the foundations upon which they rest; if upon masonry, the bases will be made square, with brackets running from the column-shafts to their edges, to distribute the load and prevent fracture of the bases through transverse stress arising from unequal pressure.

In bridges over rivers, and viaducts over estuaries, we have different foundations to deal with.

The Kent and Leven Viaducts, in the north of England, are carried on "disc" piles, the foundation being sand. These disc piles are columns with round bases, upon which are cast ribs on the under side to loosen the sand by turning them to and fro, the loose sand being blown out from under the pile by a jet of water from a tube passing through it; when a sufficient depth for stability was reached a few blows from a tup upon the head of the pile solidified the sand beneath it.

For such piers, especially in alluvial soils, screw piles

have been very largely used. The bottom column is made, or fitted, with a broad screw thread of about one and a quarter turns, and this not only serves to draw the pile down into the soil, but also supplies a good bearing surface as well. These are also very convenient for land viaducts on clay or shale strata, provided there are no boulders or pieces of rock to obstruct the progress of the screw.

CHAPTER XVI.

BEARINGS AND BED-PLATES.

IN designing the bearings for bridges provision is to be made for changes of length, caused by variations of temperature and stress, and also for movements accompanying deflection. In England the range of temperature is about 81° Fahrenheit, and in large bridges this is found to cause a variation in length of 1 inch in 150 feet. This expansion is about equal to the extension of wrought iron under a stress of 6 tons per sectional square inch. The other movement to be accommodated is the rising of the ends of the girder on the abutment caused by the deflection between the points of support.

The dimensions of the bed-plates will be regulated by the material upon which they are to rest; if they are carried by brickwork the load upon them should not exceed 3 tons per square foot, but on sandstone the load may be increased to 15 tons per square foot; if cast-iron piers carry the bed-plates, 7 tons per square inch may be applied, and 4 and 6 tons respectively for wrought iron and steel.

A bridge of 200 feet span, carrying a double line of railway, will have a maximum total load of about 4½ tons per lineal foot, and this will put upon each of the four bed-plates a pressure equal to $(4 \cdot 5 \times 200) \div 4 = 225$ tons; this would

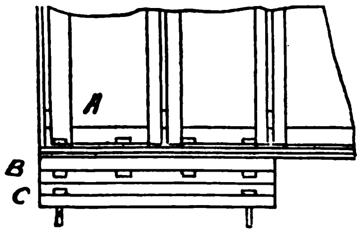

Fig. 99.

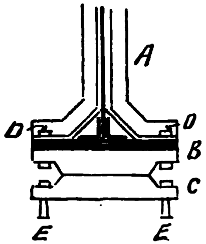

Fig. 100.

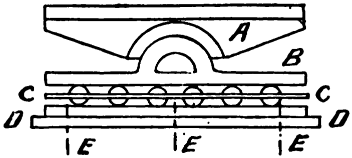

Fig. 101.

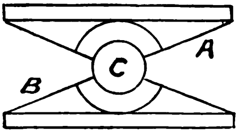

Fig. 102.

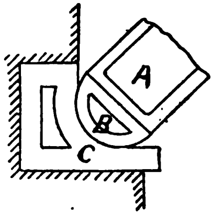

Fig. 103.

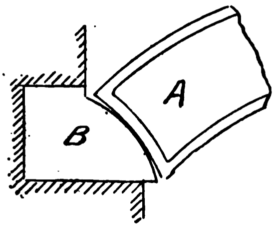

Fig. 104.

require a bearing surface on stone of 15 square feet, say 5 feet long by 3 feet wide; if such a pier were built in brick, the load would have to be distributed over it by large bed-stones.

The simplest combination of bed-plates is shown in Fig. 99 in side elevation, and in end elevation in Fig. 100. To the bottom of the end, *A*, of the main girder is bolted a cast-iron plate, *B*, by bolts, *D D*; the underside of this plate is accurately planed, and rests upon the planed upper surface of a bed-plate, *C*, which is secured by holding down bolts *E E*. The top plate slides upon the lower when variations in length occur, and the lower plate should be longer than the upper one to allow for this movement. This arrangement is commonly used for short spans, as there is no provision for deflection movements. In order that the bearing may be practically uniform, two or three layers of felt are sometimes placed under the bed-plate, *C*; this will yield sufficiently to accommodate the slight rise of the girder end. These expansion plates are, of course, only required at one end of the girder, the other end may be firmly bolted down to the pier.

An expansion and rocking arrangement is shown in Fig. 101. The plate *A* is bolted to the bottom of the girder as before, but it has on its underside a cylindrical channel truly shaped out to fit a convex projection upon the top of the under plate *B*. This plate has its underside truly planed to rest upon small rollers as shown, which rest upon the bottom bed-plate, *D*, which is secured by holding-down bolts, *E E E*. A frame of light bars, *C C*, surrounds the rollers, with holes in its sides to receive their end pins and keep them in their proper relative positions. The rollers will vary from 2 to 6 inches in diameter according to the size of the girder supported by them.

Another form of rocker is shown in Fig. 102. Here the top and bottom plates both have cylindrical recesses into which fits a pin, *C*, upon which the end of the girder can rock; the periphery of the channels in the plates *A* and *B* will be made about one-third of a complete circle to allow the necessary freedom of movement. Rockers will be required at both ends of the bridge, and should be made of Bessemer or Martin-Siemens steel. This is more easy to make true than the arrangement in Fig. 101, as the pin can be turned in an ordinary turning lathe, and the channels in plates *A* and *B* trued up in a boring machine.

If the bridge has an iron or steel flooring it will expand and contract in its width as well as its length, so that its path of expansion is along diagonal lines joining its opposite corners, and in this direction, therefore, the bed-plate should be set, otherwise there will be lateral sliding upon the rollers which may tend to jam some against the framing and damage it.

The main girders are made with a slight rise or camber, so that when deflected the line of the bottom flange shall not drop below a straight line drawn from one end to the other; the amount of this camber is usually made 1 inch at the centre for every 40 feet of clear span.

Arches, as well as girders, vary their dimensions with change of temperature, but they do this by rising or sinking at the crown. This will cause some slight movement at the haunch which may be provided for as shown in Fig. 103. The end, *A*, of the arch has a convex casting, *B*, bolted to it, and this ends in a cylindrical socket in a casting, *C*, which is built into the impost of the abutment.

I have seen the device shown in Fig. 104 applied to an arch of about 120 feet span, but in my opinion it is very objectionable, and I only introduce it here to point out

its disadvantages lest the example referred to should be followed by any of my readers.

The end *A* of the arch is made slightly concave, and it rests against a casting, *B*, which has a convexity of a slightly smaller radius upon which it rolls. Now, in this arrangement, it is obvious that the bearing surface brought into action is a very narrow strip depending for its width upon the amount of compression of the materials at the line of contact, and this may be so small that the ultimate crushing strength of the material may be closely approached.

An analogous contrivance has been used on some Australian railway girder bridges, the bearing-plates at the ends having convex under surfaces which rest upon flat bedplates; the same objection applies here as in the last case.

CHAPTER XVII.

MATERIALS AND WORKMANSHIP.

THE greatest amount of care bestowed upon calculating and designing a bridge will be wasted if the material used falls short of the standard adopted as a basis, therefore the preparation of the specification is only equalled in importance by the means to be taken to ensure its faithful observance.

It is necessary to guard against excess of any sort in drawing up a specification of materials and workmanship so that nothing shall be specified which is not easy of attainment; first, then, we must ascertain, if we do not already know, what class of material is available in the district in which the work is to be executed—it is useless to specify iron with a tensile strength of 24 tons per sectional square inch if none of higher tenacity than 22 tons can be obtained; and on the other hand if the work lies in a district where 25-ton iron is usually turned out, it is wasteful not to take advantage of the higher strength to reduce the weight of metal in proportion.

I have tested many samples of wrought-iron plates and bars which have reached a tensile resistance upwards of 25 tons per sectional square inch in both Staffordshire and Yorkshire, and the uniformity of iron is more reliable than that of mild steel.

According to Board of Trade regulations, in bridges carrying railways cast iron must not be used under any but compressive stress, so that it is practically limited to columns and arches. Its breaking strength is not to be less than three times the dead load plus six times the maximum live load. Cast-iron struts have been used in triangular-webbed girders in conjunction with wrought-iron tie-bars, but as the expansion rate is not the same for the two metals such a combination is liable to internal stresses in addition to those caused by the load, and there are no means of calculating the intensities of such stresses, therefore it is safer to keep the work all in one material. The same authority fixes the maximum stress on wrought iron at 5 tons, and on mild steel at 6½ tons per sectional square inch.

Bridge steel is usually specified to have an ultimate tensile strength not less than 28 nor more than 32 tons per sectional square inch; a higher limit is fixed, as otherwise the steel supplied might be too hard for girder work. For wrought iron a minimum limit only is specified, and if the Board of Trade working strength is adopted, the ultimate tensile strength should not be less than 22 tons per sectional square inch.

Tests of extension, and contraction of ruptured area, are also used; mild steel should extend by 20 per cent. in a length of ten inches before rupture and show 25 per cent. reduction of area, and wrought iron should have an extension of 10 per cent. before fracture.

Cast iron should show a grey fracture, and not break under a tensile stress less than 8 tons per sectional square inch. In transverse stress a cast-iron bar 3 feet span, 2 inches deep and 1 inch wide, should not break under a central load less than 25 cwt.

In wrought-iron work the stresses should always run in

the direction of the length of the plates, as the tensile resistance to cross stresses is less than in the longitudinal direction—the direction of the grain as it is termed.

Test pieces should be taken from every charge of wrought iron and steel, and in cast iron, test bars are to be supplied from each charge for experiment.

The effects of manipulation upon the materials must be enquired into. Punching damages both iron and steel, and where it is used the holes should be broached out afterwards to remove the deteriorated parts. The most satisfactory course, however, is to drill the holes out of the solid, and if suitable multiple-drilling machines are used, I do not think drilling costs any more than punching.

Then there is another great advantage with drilling, which is that a number of plates can be cramped together and the rivet-holes drilled through the whole thickness at one operation, which will ensure that the rivet-holes have their axis common; when the plates are either punched or drilled separately there is the chance of some of them being slightly out of centre, and wherever this occurs a ridge will be formed on the rivet, and this will weaken it, for it will be as if it had commenced to shear.

In works of any magnitude the riveting up is generally done by hydraulic riveters, which give a better result than hand riveting, because the rivets are closed much more quickly, and so do not have time to cool before the head is completed, and the result is that in contracting as they cool, they draw the plates very firmly together, and the more firmly the plates are pressed together the greater will be their frictional resistance to moving one upon another.

The limit of the pressure that can be thus got upon the plates will be equal to the sum of the cross-sectional areas of the rivets multiplied by the elastic limit of the rivet-iron

or steel. This elastic limit for steel is about 60 per cent. of its ultimate strength, and in wrought iron about 30 per cent. of the breaking load.

The rivets are to be so proportioned that the heads will not strip off or break under a stress less than that necessary to pull them asunder across the body, and in the same manner the heads and nuts of bolts are to have a holding strength equal to that of the body of the bolt at the bottom of the thread.

It is very important that the threads of bolts and nuts should be accurately cut; in former years we specified that they should be chased in a lathe, as the screw-cutting machines then only partly cut and partly squeezed up the threads, which often showed a ridge at the top of the thread. It is obvious that a thread so formed has very little strength, in fact not so much as a waste turning from a lathe. Merchant bolts should never be used for bridge work, and none should be passed which do not show a clean-turned surface over both body and thread, especially the latter. If the nuts are truly made no washers are necessary under them, and none should be permitted for ordinary connections; bolts and nuts will of course never be used where the work can be riveted together.

In some of the colonies work which would here be riveted has to be bolted together, because native labour only is obtainable, and in such cases the bolt ends should be lightly riveted over the nuts to prevent their becoming loose.

Large pins used in triangular-webbed girders and for chain suspension bridges are made with square threads cut in a lathe, and for the best work the nuts are secured from turning, either by a pin passing through each or a cotter passing through the bolt beyond the nut. The pin fixture is the better of the two.

These large pins should be accurately turned and the holes in the flange-plates accurately bored out to fit them, for any loose fit means rattle, and, ultimately, failure of the structure.

I will now deal generally with the fitters' work on bridges, and what is necessary to prepare the parts for connection.

Now, according to the width of the girder flange it is made of bars or plates. Bars are rolled up to 12 inches in width, but the technical difference between a bar and a plate is that a bar is finished in the rolling mill, but a plate has to have its edges sheared after leaving the rolls; so it is obvious that if we choose to specify it we can have any flange-plates rolled as plates, although their widths are under 12 inches.

If only one plate is required for a flange a bar is suitable, but if two or more thicknesses are required, they should be plates, as they will require planing along the sides to bring them all to the same width; they will also require planing at the ends.

The allowance for planing flange-plates varies in different works, and also in different methods of ordering. In works executing only orders for first-class work it is usual to allow $\frac{1}{4}$ inch for planing on sides and ends of plates; in bridge works connected with rolling mills, or having their own mills, the plates are ordered to nett dimensions from the bridge department, the forge managers putting on the margin for planing, which will depend upon how truly they can shear the edges of the plates.

So long as the widths of the plates are equal the actual width of flange will not matter to $\frac{1}{32}$nd of an inch; but the lengths should be dead true, certainly in the plates to be used in the compression flange, for if the joints in that are

open there are only the cover-plate rivets to carry over the stress. In case such bad joints should occur I always put in enough rivets to carry the stress, but this involves a waste of money that need not be incurred were we sure of getting a true fit at the joints.

Fig. 105 shows a cross-section of a plate-girder 3 feet deep; *A* and *B* are the top and bottom flanges, *c* the web, and *D D D D* the angle-irons connecting the web with the flanges. Tee-iron stiffeners *E E* are used to give rigidity to the girder, that is, to stiffen the web *C*, and to resist twisting in the top flange *A* and its angle-irons.

Riveting through the web and the vertical limbs of the angle-irons is sufficient for the web stiffening, assuming that the stiffeners are joggled to fit properly the draught of the main angle-irons, but the ends of these stiffeners must be cut to fit tightly the inner sides of the main angle-irons, otherwise no support will be given to the flanges.

Fig. 106 shows a cross-section of a rolled girder *A*, with stiffeners *B B*. Now, from the form of the section these rolled girders always have thick webs in comparison with plate-girders; so here especially is it necessary that the stiffeners should be fitted to the draught of the flanges, which is about 8 degrees beyond the square for the steel girders rolled in England. As the rolled girders will—from one mill—always have the same draught the stiffeners can be machined to a gauge, there will be no need for hand chipping by the fitter.

Fig. 107 shows a part longitudinal section of a bridge-floor with a distributing girder made with flange-plates passing above and below the cross-girders *A A*. These flange-plates *B B* are riveted to the flanges of the cross-girders, and connected between them by a plate-web and angle-frames *C C*, which frames are also riveted to the webs of

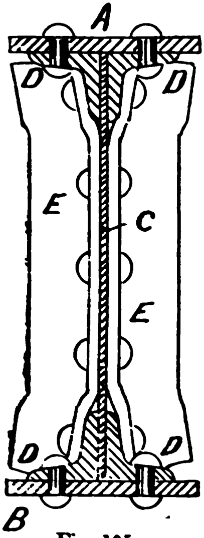

Fig. 105.

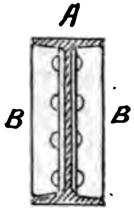

Fig. 106.

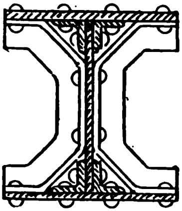

Fig. 108.

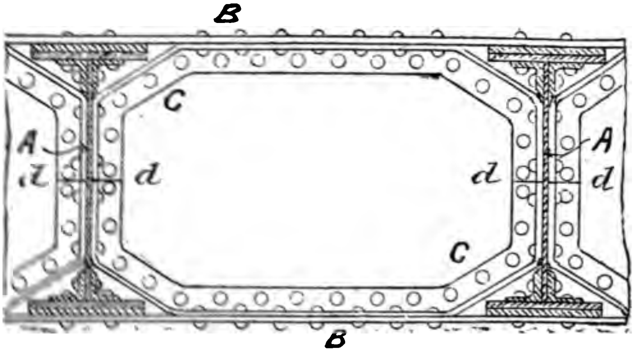

Fig. 107.

the cross-girders. An example of this construction occurs in the bridge which carries the Great Northern Railway over the Leeds and Liverpool Canal at Shipley, Yorkshire.

In such work it is essential that the discontinuous web shall fit truly to the flange-plates and to the cross-girders, and therefore, that the angle-iron frames should all be alike, in fact interchangeable. It is not necessary that each frame should be in one piece, it may be made in two, the divisions being at *d d*; then each angle-bar can be bent in a hydraulic press in dies, which will ensure similarity.

Fig. 108 shows a cross-section of a broad-flanged girder with cranked stiffeners. If there are only a few of these it may not be advisable from an economical point of view, to make a special die for cranking them, and in that case, they will be smithed, but he will be an exceptionally good smith who makes two exactly alike in measurement; if a fair number, say 60, are required, it will be cheaper to make dies and press them.

Wherever the hydraulic press can be used for quantities of work, as against the hammer, there is about 30 per cent. saving in cost of labour and machining, and that is why better work can be turned out in large yards, at a lower cost, than that incurred in smaller establishments.

I will now pass to work in cast iron; the workmanship here commences with the pattern-maker, and finishes as far as the factory is concerned in the machining shop, where the caps and bases are turned or planed, as occasion may render most convenient. A great deal of responsibility rests upon the pattern-maker, for if he has a drawing given to him showing sections that will not probably stand the stress of cooling, he should get permission to alter them; but assuming the drawings to have been made by a competent engineer, he will use his discretion about draught on flanges

and similar matters, so that the pattern may leave the sand easily, and leave it clean, as to surface and fillets. I have only lately, well in the course of the past year, known cast-iron stanchions to weigh 12 per cent. more than their calculated weight, and this was due to the use of roughly made patterns which had to be knocked about to get them out of the sand.

Columns and stanchions should be cast upright, so that no air or gas from the sand can settle in their substance, and the head of metal above should be sufficient to receive all the bubbles generated below. In the same way the segments of a cast-iron arched rib should be cast on end—the part nearest the abutment at the bottom, with an ample "head" at the upper end.

It however happens, unfortunately, that not many foundries have convenience for casting long columns vertically, and therefore there is always a liability to sponginess in the body of the casting.

The testing of a column for sponginess or inequality of thickness, is an operation for which facilities are seldom available; thickness may be determined by drilling holes through the sides, but this—although the holes may be plugged—must weaken the casting.

The only practicable test of uniformity, is to apply transverse stress to the column in four different positions, and note the deflection in each. To carry this out, the column is laid on supports, its base on one and its cap on the other, in such manner that its axis is horizontal; a central load equal to $\frac{1}{3}$ of its calculated breaking weight is then applied; after the deflection has been noted, the column is turned one-quarter round, and the same load again applied, and the deflection noted; the same test is applied through two more moves of the column; if the

deflections are equal in every test, the homogeneity of the column is perfect, but this is very seldom found to be the case; if, however, there is not a variation of more than 5 per cent. in the deflections, the column may be passed as sound.

In regard to the machining of castings, when true faces are required, they must be planed or turned—the emery wheel is not to be used for such work, and the caps and bases of columns must be faced square to their lengths.

Iron is an exceedingly oxidizable metal and therefore liable to rapid deterioration by corrosion and rust. The slightest break in a protective coating serves as a starting-point whence rusting can extend along the metallic surface and into the substance of the metal as well.

The molten iron used in making castings takes on a skin of sand which, if unbroken, forms a very good protection against atmospheric influences, but wrought iron and steel have no such natural covering, and therefore must be painted with some resisting matter to exclude the ravages of rust. In some materials—aluminium for instance—a coating of rust forms in itself a protection, but this is not the case with iron, for the red oxide will give up part of its oxygen to the metal behind it and take up more from the moisture in the atmosphere and so continue to extend the penetration of the material. If, however, this rust, or "minium" as it is called, is made up into a paint with oil, it makes a very useful coating and is commonly used for iron structures in the first instance next the metal, any decorative colours that may be desired being put on afterwards. In the application of this first coating the greatest care must be taken to cover every part and leave not the smallest spot open to the air, or rusting will inevitably set in.

Iron is also protected by coating it with tin, or zinc, the latter by a process miscalled "galvanizing," as electricity has nothing whatever to do with it. The iron sheets having been thoroughly cleaned are put in a pickle of commercial muriatic acid to remove any traces of oxide, and are then immersed in molten zinc; when lifted from the zinc bath any excess of zinc runs off, leaving a thin coating of zinc on the surface of the iron sheets; sometimes a little tin is added to the zinc, which forms a spray pattern on the iron. The molten zinc is covered with a layer of sal-ammoniac to prevent waste.

In works of any magnitude or importance it is usual to appoint an inspector to watch the work during its execution, and he, of course, must be a thoroughly trustworthy man, and above all things, beyond the suspicion of being workable by the manufacturers, to secure which he should be liberally paid. It will be his business to attend the testing of all materials and keep an attentive eye on the quality of the workmanship put into the job.

All manufactured material is liable to flaws, and a simple means of detecting the existence of such is a great desideratum. Saxby's magnetic test is convenient for this purpose but it is not so widely known as it should be.

If a bar of iron is placed in an east and west direction, magnetically, and a small compass needle slowly passed in front of it, the needle will not be disturbed if the iron is homogeneous throughout, but if there is any break in the continuity of the material the needle will, on reaching the point where it occurs be deflected. At the time of this discovery a great many tests were made in the dockyards of Chatham and Sheerness. Several bars were examined by the compass, and the points at which it deflected were marked by chalking or tying a string round the bar; then

the bars were put in a testing machine and broken, and in all cases the bars broke at the places so indicated. In one test bar of 1-inch square forged iron, a piece of steel 5 inches long was welded. The needle detected the fault at about the centre of the piece of steel; the iron bar was $12\frac{1}{2}$ inches long.

From these experiments it is deduced that the magnetic needle is capable of detecting the position of any solution of continuity, or flaw in iron bars of symmetrical form, though, of course, it would not be applicable to articles of irregular form.

In testing for tensile resistance, extension of length and reduction of sectional area are taken into consideration. The piece to be tested should be planed in the centre of its length to exact dimensions, and two fine centre punch marks made 8 or 10 inches apart, the distance of these apart after the fracture will show the extension; to find the elastic limit of the material the load must be repeatedly applied in increasing quantities until, when removed, the bar does not resume precisely its former length, we then know that the limit of elasticity has been passed.

If such tests are not judiciously conducted, very misleading results may be reached, especially when the stresses are put upon the bars by means of hydraulic pressure. The tearing asunder of wrought metal takes an appreciable time after fracture has commenced, and it is, therefore, possible to run the stress somewhat higher to expedite a rupture that, with more time for action, would be accomplished under a smaller stress. Now, directly the cohesion of the metal begins to fail, a sound becomes noticeable, and as soon as this sound is heard the application of further stress should be stopped, and time allowed to show

whether the test bar will break under the load already put upon it.

An indication of uniformity of metal—or the contrary—may be found by rubbing the sides and edges of the test bar with chalk, and watching to see how it falls off when the bar is stretched; if it falls off evenly the material is homogeneous, but if it cracks off on one side sooner than the other, it shows that that side of the bar is more extensible and therefore of less density than the other. In such a case the bar will more probably commence to tear asunder on one side, the rent passing across to the other; if, however, the texture is uniform, both edges will tear at once, and this will be a stronger sample than the other, because it will be exerting its whole resistance at once instead of being torn through in detail.

It is not to be assumed that permanent extension always involves a loss of strength, for in some cases the contrary effect is brought about, an example is found in iron and steel wire, which after the wire-drawing operation shows a greatly increased tensile strength, but it does not follow that bars permanently stretched, without at the same time undergoing lateral pressure, will receive any accession of strength. If a bar or plate has been badly worked, or cooled too quickly in the rolls, it may well happen that its molecules will not have fallen into their normal positions, and under such circumstance there will be internal stresses which are not ascertainable, and it is reasonable to think that a heavy stress, exceeding the elastic limit, may allow these molecules to so re-arrange themselves that the internal stresses disappear and the strength of the whole sectional area becomes available.

The amount of deflection caused by a given load is *frequently* used as a test of the strength of materials. It is

shown by calculation, and proved by experiment, that while the elasticity of a bar under transverse stress remains unimpaired, the amount of deflection will be in direct ratio to the load applied—other conditions of course remaining unaltered.

The modulus of elasticity is very variable and difficult of determination by any direct force, therefore the elasticity of any particular parcel of metal is easiest found by experimenting with transverse stress.

To find the modulus of elasticity only light loads should be used so as to keep well within the limit, for as that is approached closely irregularities are liable to appear. This being determined, the limit of elasticity is found by loading the bar till it takes a "permanent set"; it does not follow that it will retain the whole deflection impressed upon it, but it will not return immediately to exactly its original form, nor after a lapse of time, but sometimes an apparently permanent set will be recovered if no other load is applied for a while.

When the amount of this deflection is ascertained we can find what deflection should be allowed as the limit of safety, for it is obvious that the constant repetition of a load that produces a permanent deflection will ultimately lead to destruction.

The great advantage of the deflection test is that it can be applied without involving the destruction of the work tested, and is therefore applicable to a completed structure, and will show if any part is defective by irregularity in the curve of deflection, such part—which may have a flaw in it—can then be removed and replaced by sound material.

Dealing with a completed girder is a very different thing from experimenting upon a solid bar, or a solid cast-iron, or *cast-steel girder—in* the latter the permanent set can only

indicate molecular distortion, but in the girder there is another kind of "set" which is due to defects in workmanship.

It is very easy to conceive that when a girder has been riveted and bolted together on a platform that there will be no bearing stress upon the rivets and bolts to draw them up to their bearings, and it is exceedingly improbable that their fit to the holes they are intended to fill will be identical throughout the work; therefore when the supporting platform is removed, and the girder has to carry its own weight, the joints will most likely draw to some slight extent—enough to give the whole girder a slight permanent set, and when, in addition to its own weight, it has a useful load to carry, this set will be increased, for the resistance of friction of the riveted plates may be further overcome, until the solid bearing of the rivets and bolts in their holes arrests all further movement except that permitted by the elasticity of the solid metal.

This condition being reached, the girder has assumed a form to which it will return after every passage of a load, if its proportions have been properly adjusted to the loads to be carried.

If, however, it is observed that a gradual increase of permanent set is occurring, this is an indication that the elastic limit of the material itself is being exceeded, and the work must be strengthened, otherwise failure will occur.

The class of workmanship will have a most marked effect upon the strength of a structure, for it is obvious that in punched work, even where the holes are broached out afterwards, there is no certainty of all the rivets getting so uniform a bearing as when the holes are drilled out of the solid, and through all the plates at once that are to be riveted together; in this latter case we have at the joints

l strength of all the rivets and the friction between ıtes in addition, whereas in the former case we may he strength of some of the rivets aided by the plate ı, or we may get the resistance of most, or perhaps in xceptional case all the rivets, but with the loss of the ı of the plates, which must have slipped slightly to he rivets to take their bearings.

CHAPTER XVIII.

ESTIMATION OF WEIGHTS.

WHEN a bridge has been designed according to the first assumed loads, it is necessary to calculate its weight in order to see if a sufficient addition to the previously known loads has been included in calculating the stresses.

The calculation of the weights of iron and steel work is a simple matter; a plate of wrought iron 1 foot square, and $\frac{1}{4}$-inch thick, weighs 10 lbs.; therefore, a bar 1 inch square by 1 yard long weighs also 10 lbs., or $3\frac{1}{3}$ lbs. per lineal foot. Mild steel weighs 2 per cent., $\frac{1}{50}$, more than wrought iron. Cast iron weighs 38 lbs. per square foot, 1 inch thick, or $3\frac{1}{6}$ lbs. per square inch of section per foot run. The weights of different brands of cast iron vary widely, but this is the weight that should be specified for bridge iron. Fillets, stiffeners, brackets, and other similar parts are taken off in cubic inches, and as there are 144 square inches in a square foot, the weight of 1 cubic inch is 38 lbs. $\div$ 144 inches $=$ 0·263 lb.

The multiplier to be used will depend upon the character of the work to be dealt with; thus, if we have a girder of uniform section throughout, we should work on the sectional area. To take a simple example, let the weight be determined of a girder, 40 feet long $\times$ 4 feet deep $\times$ 15 inches wide on the flanges, and let each flange consist of two plates

each ½-inch thick; let there be tee-iron web stiffeners, 6 inches × 3 inches × ½ inch, every 4 feet of the length on both sides of the web. There will be required a cover joint plate, 6 feet long, for each flange. The web is to be taken as ⅜-inch thick, and the angle-irons joining it to the flange-plates as 3½ inches × 3½ inches × ½ inch. Each of these will require an angle-iron joint-cover, 3 feet long and of equal sectional area. These covers are rolled with round backs instead of square corners, so as to fit into the roots of the angle-irons to be connected. There will be the end angle-iron, and end-plates ½-inch thick, and the bed-plates 2 feet long, each under the ends of the girder—these ¾-inch thick and the rivet-heads also to be allowed for. The tee-iron stiffeners and end angle-irons will be taken as having their ends joggled over the main angle-irons, so no packing-bars will be required under them.

The web-plates will not make up a length of 40 feet, as ½ inch will be taken up at each end by the end-plates, and in other parts there will also be slight reductions, but these are not appreciable so far as our purposes are concerned, and therefore the gross dimensions will be taken.

I think I shall make this calculation of weights clearer by taking the items in detail, than by scheduling them without further explanation, and shall therefore adopt the former course.

We will take first, the running section which comprises the web-plates, the flange-plates, and the main angle-irons; the vertical sectional area of the girder will be,

Web, 48 ins. deep × ⅜ in. thick . . .	= 18	square inches
Flanges, 4 plates, 15 ins. wide × ½ in. thick	= 30	,, ,,
Angle-irons, 4 bars, 3½ ins. × 3½ ins. × ½ in.		
= 4 × 6½ ins. × ½ in.	= 13	,, ,,
	61	,, ,,

The weight per foot of this section will be 61 square inches $\times$ 3·3$\dot{3}$ lbs. $\doteq$ 203·3$\dot{3}$ lbs. per lineal foot, and the total weight

$$= 61 \text{ feet} \times 203{\cdot}33 \text{ lbs.} = 12{,}403 \text{ lbs.}$$

decimals are dropped as inconsiderable.

There will be four angle-iron cover bars of sectional area equal to that of the main angle-irons, which is, for each $\{3{\cdot}5 \times 3{\cdot}5 - 0{\cdot}5\} \times 0{\cdot}5 = 3{\cdot}25$ square inches.

This will give a weight per foot run $= 3{\cdot}25 \times 3{\cdot}33 =$ 10·8$\dot{3}$ lbs., and the total weight of the angle-iron covers will be,

$$= 4 \times 3 \text{ feet} \times 10{\cdot}83 \text{ lbs.} = 130 \text{ lbs.}$$

There will be four end angle-irons, of which the weight will be,

$$= 4 \times 4 \text{ feet} \times 10{\cdot}83 \text{ lbs.} = 173 \text{ lbs.}$$

The weight of the two end-plates and of the cover-plates for the flanges may be taken by surface measurement, two end-plates at 4 feet long each and two cover-plates 6 feet make a total length of 20 feet, the width is 15 inches $=$ 1·25 feet and the thickness $\frac{1}{2}$-inch, giving 20 lbs. per square foot, so the weight of these plates will be,

$$20 \text{ feet} \times 1{\cdot}25 \text{ feet} \times 20 \text{ lbs.} = 500 \text{ lbs.}$$

The sectional area of the tee-irons is $\{6'' + 3 - \frac{1}{2}\} \times \frac{1}{2}$ $= 4{\cdot}25$ square inches, and therefore its weight per lineal foot $= 4{\cdot}25 \times 3{\cdot}3\dot{3} = 14{\cdot}17$ lbs. The length of the girder is divided into ten bays, therefore there will be nine tee-irons on each side of the web, each tee-iron being 4 feet long; the weight of these will be,

$$\text{No. } 18 \times 4 \text{ feet} \times 14{\cdot}17 \text{ lbs.} = 1{,}020 \text{ lbs.}$$

These weights can now be added together and five per cent. allowed for the weight of rivet-heads, this being found ample in practice for work of this class.

Web-plates, flange-plates, and main angle-irons	=	12,403 lbs.
Angle-iron cover-bars	=	130 ,,
End angle-irons	=	173 ,,
End-plates and flange cover-plates	=	500 ,,
Tee-iron stiffeners	=	1,020 ,,
		14,226 ,,
Ten per cent. allowed for rivet-heads	=	1,422 ,,
		15,648 ,,

= 6·54 tons.

When the weights are taken off for estimation of cost the different forms of iron must be classified, thus the plates will be brought together for one item, which may be subdivided if the sizes vary sufficiently to have different values per ton; in short the metal must be classified according to the quotations for different sections, lengths, and widths.

I shall not touch upon the question of cost, it would be useless to do so, for that which is written to-day as to prices may be wrong to-morrow, and an estimate of cost of labour this week may be upset by a strike next week; therefore printed information would be misleading, in this direction, to any who relied upon it.

However, leaving the prices alone, I may indicate the course to be pursued in preparing a bill of quantities for pricing.

In such bills the nett weight of the finished work will be inserted, but waste must be charged for, and therefore the draft schedule will be different from that of the finished work, wherever waste occurs.

Wrought-iron bridges have been constructed with flange-

plates tapered on plan; I remember some that were made for Russia in 1870 on such designs; the plates were rolled more or less parallel and sheared to the required curve, but this made a considerable waste, which was of course taken into consideration in the price. In ordering plates from the mills an allowance is usually made for planing edges for best quality work about $\frac{3}{16}$ to $\frac{1}{4}$ inch on all sides, this will give a percentage which must be added to the prime cost as applied to nett weights.

On the ends of bars there is also an allowance for planing which will be treated in the same way. The amount of such percentages should be accurately ascertained in each case; averages are guess-work when applied to individual works.

The engineer who is capable of designing a bridge should also be able to calculate its weight to within 1 or $1\frac{1}{2}$ per cent., but to do this he must be conversant with the customs of the rolling mills.

There are two ways of letting a contract for bridge work: one, so much per ton; the other a lump sum for the whole work. Which course is followed will depend upon circumstances outside the scope of this work, but in either case the engineer must take care that his clients have full value for their money.

Such heavy work as that with which we are dealing is generally paid for upon the basis of calculated weights, so precautions should be taken to secure, as far as possible, that the bars and plates used should come up to their calculated weights; although it is not convenient to weigh a large girder, yet plates and bars may be weighed before they are used to ascertain if the sections come up to the weights specifi[illegible] Instead of specifying sizes only, weights per f[illegible] marked upon the drawings; thus instead of

marking an angle-iron $3\frac{1}{2}'' \times 3'' \times \frac{1}{2}''$ it can be marked $3'' \times 3'' \times$ 10 lbs. per foot. A tee-iron similarly could be figured $6'' \times 3\frac{1}{2}'' \times$ 15 lbs. per foot, instead of $6'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$. The same course may also be taken with plates and other sections.

Specifying weights per foot also puts the quantities on a fixed basis, which sizes alone will not, as there are differences in different mills; tee, angle, and channel-iron and steel, cannot be rolled without some draught or taper to allow them to come freely from the rolls and this will vary in different districts with the different qualities of iron produced in each.

In published tables of weights the sections are treated as absolutely rectangular, with no rounding off of arrises or filling of internal angles, and therefore although these weights are useful for rough estimates, contract drawings should be marked more definitely, and especially when the manufacturers invited to tender are not only spread over England and Scotland, but also established in Belgium and Germany.

CHAPTER XIX.

BRICK, MASONRY, AND CONCRETE PIERS AND ABUTMENTS FOR IRON AND STEEL BRIDGES.

In the majority of cases in ordinary practice iron and steel bridge girders rest upon brick or masonry piers, or abutments, or on concrete columns encased in iron; it is therefore necessary that the iron and steel bridge designer should be acquainted with the principles and practice upon which the details of these parts of the structures are based.

Bridges carried by girders, or by tied arches, put only a vertical load upon the supports, but very often those supports have some other duty in addition to carrying the bridge. The piers of a single-span bridge carrying a railway over a public road will usually have also to act as retaining walls to the banks of earth behind them, upon which the railway is made. It is therefore necessary to consider the conditions upon such piers, independently of the bridge load. I may mention that the dead weight of the bridge will increase the stability of the pier, but in case of renewals or alterations in the future, the wall or pier should be made of sufficient stability to uphold the earth behind it, without such adventitious assistance.

We must now determine the conditions of maximum stress upon a retaining wall, assuming that it is practically solid.

Each kind of soil or earth has a slope at which it will

stand, without support from artificial works, but if the bank of earth is to terminate in a vertical plane, a wall is required to support it. The angles of natural slope of different materials are shown in the following table :—

INCLINATION OF NATURAL SLOPES OF EARTHS, &c., TO THE HORIZON.

Gravel	average	40 deg.		Shingle	average	39 deg.
Dry sand	,,	38 ,,		Rubble	,,	45 ,,
Sand	,,	22 ,,		Clay, well dried	,,	45 ,,
Vegetable earth	,,	28 ,,		Wet clay	,,	16 ,,
Compact ,,	,,	50 ,,				

In such a table only averages can be given, and the slopes proper to any particular districts must be found by inquiry or observation.

The duty of the retaining wall now under consideration is to support a prism of earth which would, without such support, slip down. This prism acts as a wedge between the earth plane upon which it rests and the back of the retaining wall. It cannot actually slip without bringing into action the frictional resistance of its earth bed, and also the back of the wall; but the latter is never taken into calculation, as the earth may be in very light contact with the back of the wall, in which case it might start to slip before the vertical friction could act to retard its motion.

Fig. 109 shows a vertical section of a bridge pier and the earth bank behind it; *a e* is the top of the bank, and *f d* the ground surface upon which it rests, both taken as horizontal. Let *c b* be the natural slope of the ground—that to which it would weather if left unsupported—*f e a c* is the section of the pier. Now, it does not follow that the slipping of the whole prism *a b c* will cause the greatest pressure upon the pier, for although a part slipping would have less weight, yet the wedge would have a more acute

angle, so we must find out what relation the prism of greatest pressure bears to that of the natural slope.

The angle of natural slope is also the limiting angle of friction, and this is intimately concerned in this investigation, because when the earth prism is on the point of slipping the frictional resistance of its bed comes into operation, and the reaction from that bed is not normal to it, but at a higher angle equal to the natural angle of repose set off from the normal to the plane upon which the earth prism would slip.

Let *c h* be the line of slip corresponding to the maximum thrust on the back of the pier, and for weights let one foot be taken along the face of the pier.

Find the centre of gravity *g* of the triangle *a c h* in the usual way by bisecting two sides and drawing from the points of bisection straight lines to the opposite angles, the intersection of these lines gives the position of *g*.

From *g*, draw the vertical line *g i*, cutting the line of slip *c h* in *k*; from *k* draw a straight line *k l* at right angles to *c h*; now if there were no friction along the line *c h*, *l k* would be the direction of the reaction of the earth supporting the prism *a c h*; but when this prism of earth is on the point of slipping, the frictional resistance of the surface comes into action, and alters this direction by an amount equal to the limiting angle of friction, or angle of repose. Therefore, make the angle *l k m* equal to the angle *b c d*, then *m k* will be the direction of the reaction of the earth upon the prism *a c h*.

On the vertical line *g i*, and from the point of intersection *k*, mark off, to any convenient scale, *k n*, equal to the weight of the prism of earth *a c h*, and complete the parallelogram *k o n p*, then will *k o* represent the horizontal thrust against the back of the wall.

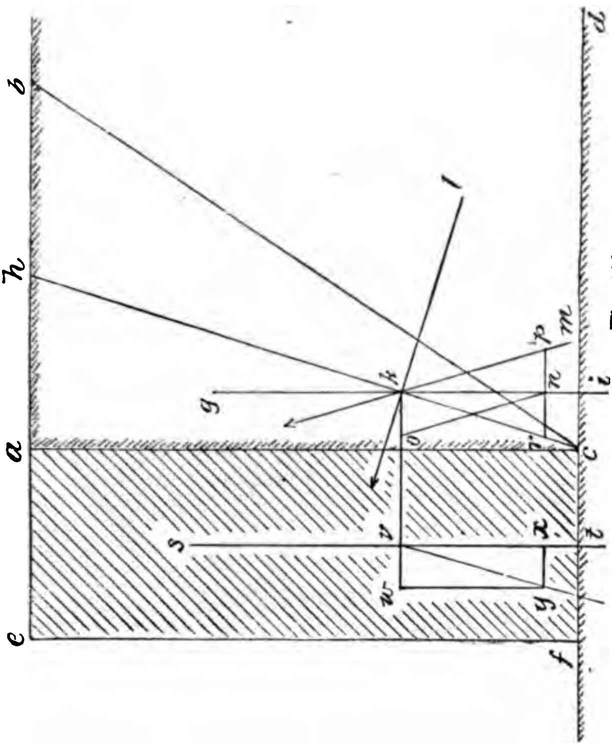

Fig. 109.

The next step is to find the position of the line $c\ h$ that corresponds to a maximum value of $k\ o$ or $n\ p$.

Let H = the height of the wall in feet, and w = the weight of the earth per cubic foot, then the weight of the prism of earth will be,

$$= H \times \frac{\overline{ah} \times w}{2}$$

Because $g\ l$ is vertical, and therefore parallel to $a\ c$, therefore the angle $n\ k\ c$ = the angle $a\ c\ h$; $k\ l$ is drawn at right-angles to $c\ h$; the angle $l\ k\ m$ is drawn equal to the angle $b\ c\ d$; $c\ d$ is horizontal, therefore $a\ c\ d$ is a right-angle. As the angles $l\ m\ k$ and $b\ c\ d$ are equal, and $n\ c\ k$ is equal to $a\ c\ h$; if $l\ k\ m + n\ k\ c$ be taken from the right-angle $c\ k\ l$, and the angles $b\ c\ d + a\ c\ h$ be taken from the right-angle $a\ c\ d$, the remaining angle $m\ k\ n$ will be equal to the remaining angle $h\ c\ b$.

Produce the horizontal line $p\ n$ until it meets the line $h\ c$ in the point r, then the right-angled triangle $k\ n\ r$ is similar to the triangle $c\ a\ h$, and its area varies as that of $c\ a\ h$; and therefore as the weight of the prism of earth $a\ c\ h$; $n\ p = o\ k$.

The horizontal thrust will vary as the area of

$$k n r \times \frac{p\,n}{k\,n}$$

This variable quantity is now to be examined in order to ascertain under what conditions it, and therefore the horizontal thrust, has a maximum value.

The area of a triangle varies as its base multiplied by its height (the area being = base × height ÷ 2); hence the quantity with which we have to deal is

$$= k\,n \times r\,n \times \frac{n\,p}{k\,n}\,r\,n \times n\,p$$

The ratio of $r\,n$ to $n\,p$ corresponding to a maximum product now remains to be determined. If $r\,n$ is increased, $n\,p$ is diminished, and the converse. When the value of the product is a maximum, then the quantity resulting from diminishing $n\ p$, and increasing $n\ r$ by an indefinitely small quantity will be equal to that found by increasing $n\,p$ and diminishing $n\ r$ by the same quantity.

Let the indefinitely small quantity be represented by u; then,

$$(\overline{rn} + u) \times (\overline{np} - u) = (\overline{rn} - u) \times (\overline{np} + u),$$

whence,

$$\overline{np} \times u - \overline{rn} \times u = \overline{rn} \times u - \overline{np} \times u;$$

therefore,

$$n\,p = r\,n.$$

Because the angles $k\,n\,r$ and $k\,n\,p$ being right-angles are equal, and $r\,n = n\ p$, and $k\,n$ is common to the triangles $k\,n\,r$ and $k\,n\,p$, the angle $r\,k\,n$ or $c\,k\,n = m\,k\,n$; therefore, the equals of these angles $a\,c\,h$ and $h\,c\,b$ are also equal, and the line of slip $h\,c$ bisects the angle $a\,c\,b$ when the horizontal thrust is a maximum.

Let $h\ c$ be drawn to bisect the angle $a\ c\ b$, and the horizontal thrust determined as shown above; then draw a vertical line $s\,t$ through the centre of gravity of the wall. Produce the horizontal line $k\ o$, cutting $s\,t$ in the point v; make $v\,w = k\,o$, and $v\,x =$ the weight of the wall.

Complete the parallelogram $v\,w\,y\,x$; the $v\ y$ will be the resultant thrust on the wall; on the base line $f\,c$, this resultant should not approach the point f nearer than one-quarter of the thickness of the wall, this will give as factor of safety 2.

The horizontal thrust having been determined a formulæ

may be found for the thickness of the retaining wall or pier. The horizontal thrust has its centre of action at a point one-third of the height from the base of the wall; call the thrust T, then its overturning moment will be

$$M = T \times \frac{H}{3}$$

If w^1 = the weight of the wall per cubic foot, and t = its thickness, then the weight of 1 foot length of the wall will be $= w^1 \times H \times t$. Its centre of gravity will be in the centre of its thickness, and therefore the moment of resistance to overturning will be equal to the weight of wall multiplied by half its thickness; that is

$$= w^1 \times H \times t \times \frac{t}{2} = \frac{w^1 . H . t^2}{2}$$

If a factor of safety, 2, be taken, the moment of stability must equal twice the overturning moment, or,

$$\frac{w^1 . H . t^2}{2} = 2\,M = \frac{2\,T . H}{3}$$

whence,

$$t = \sqrt{\frac{4\,T}{3\,w^1}}$$

The question of footing areas to distribute the total load I shall take up after dealing with abutments for arches, which next require our attention.

As has been shown in the case of a tied arch, the *horizontal* thrust at the abutment is equal to the thrust at the crown of the arch, if therefore, the centre of the abutment plate is h feet above the footings of the abutment, the overturning moment from the thrust of the arch will be, taking T as the horizontal thrust,

$$M = T \times h.$$

The total height of the abutment will be much more than h, as it will be carried up to the spandril-girder, and the expression used before will apply for the moment of stability. In this case we have the pressure of earth behind the abutment assisting it to resist the thrust of the arch in front, but upon this we should not rely.

Assuming the abutment to be of the same thickness throughout its height, and using the same factor of safety as before,

$$\frac{w^1 . H . t^2}{2} = 2 M = 2 . T . h$$

therefore

$$t = \sqrt{\frac{4 T . h}{w^1 . H}}$$

It is also necessary to the stability of an abutment that the line of thrust shall not make with a line at right-angles to any longitudinal joint, an angle greater than half the limiting angle of friction, lest the masonry should slip one course upon another. In ordinary practice it is found that, with abutments giving the requisite stability, horizontal courses will meet this condition.

Solidity and uniformity of material must be insisted upon in the construction of piers and abutments in masonry; inside rough work with facing bricks is not to be tolerated in structures such as we are now considering, and shells of brickwork filled in with concrete are best avoided, as constant supervision is necessary in such a case to insure the use of properly-made concrete. In some branches of trade any kind of ashes, I might almost say refuse, is thought good enough for making cement concrete, and even then the mixing is often as inadequate as the material to the purpose sought to be fulfilled. If a little unslaked

lime gets into this mixture it swells and bursts out the walls which should confine it.

Instead of providing weight in these abutments, they are frequently built of sufficient length back from the bridge to let the line of thrust fall well within the prescribed limit of base width, and the earth bank behind runs out at its natural slope before reaching the face of the masonry.

The size of the bed-plates upon which the ends of the girders rest will depend upon the bearing capacity of the bed-stones immediately beneath them, and the size of the bed-stones will depend upon the resistance of the masonry or brickwork upon which they rest, while the area of the footings of the pier must be adapted to the resistance of the soil upon which the structure is erected.

Using 8 as the factor of safety the working strength of various stones—in tons per square foot—is given in the following table :—

Material.	Safe Load.
Red Cheshire Sandstone . . .	17 tons per sq. ft.
Derby grit " . . .	25 ,, ,,
Portland Oolite	33 ,, ,,
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Suppose we have a bridge 200 feet span, carried by two main girders, with a maximum load of 5 tons per lineal foot; the total load will be 1,000 tons—250 tons on each bed-stone. If Portland stone is used for the bed-stone, the *iron* bed-plates resting upon it will require an area of

$250 \div 33 = 7{\cdot}6$ square feet; so a plate 4 feet long by 2 feet wide would give a margin of strength.

Good red brick well laid with thin joints will carry 10 tons per foot square as a safe load; therefore, if this is used, the stone bed must be $250 \div 10 = 25$ square feet in area, say 6 feet long by 4 feet 3 inches wide. The thickness of the stone should not be less than its extension on each side of the bed-plate—13½ inches.

Care is to be taken in selecting the bricks, as common bricks will not take a 5-ton load with 8 as the factor of safety, and furthermore, the bricks must be uniform in size and cleanly shaped, otherwise the joints will be too thick. Blue Staffordshire brick is very strong, taking a safe load of 17 tons per square foot, and it is therefore frequently specified for heavy works.

If the bricks are recessed the mortar or cement used should have a crushing resistance at least equal to that of the bricks, otherwise their fully bearing surface will not be available. Neat Portland cement after about three months has a safe resistance of 30 tons per square foot, and is therefore superior to any of the bricks and equal to some of the stones. This cement is also very valuable for its tensile resistance, which enables it to hold brickwork together in a solid mass. The cement should weigh at least 110 lbs. per imperial striked bushel, and should, after seven days, withstand a tensile stress of 200 lbs. per square inch of sectional area.

The resistance of the soil must be ascertained in the locality where the structure is to be erected to find its bearing capability. From the weights of existing works we find that blue clay will bear from 1 to 1·7 tons per square foot, and yellow sandy clay from 2 to 2·8 tons; *compact clay,* 1·5 *tons*; compact sand, 2·3 to 2·9 tons;

coarse gravel, 4·4 tons; clay and sand, after settlement ceased, has borne as much as 9 tons. In some of the clayey soils of England a full load on the railway viaducts would put 6 to 8 tons on the square foot, and the very high viaducts in the stone districts put a much greater load than this on their foundations.

Suppose in the above case we find it is not advisable to put a heavier load than 3 tons per square foot upon the bearing soil, there will be two bed-plates on each pier, and therefore a total load of 500 tons, which will require $500 \div 3 = 166{\cdot}\dot{6}$ square feet of foundation surface.

If the bridge is one for a double line of railway, which is what I have in view in estimating the loads, the width of the pier on the face will not be less than 30 feet, and therefore the thickness necessary at the base will be $166{\cdot}\dot{6} \times 30 = 5{\cdot}5\dot{5}$ feet. It is better not to let the brick-work, even if made up into blocks, bear directly on the earth, a solid concrete foundation should first be put in.

Concrete piers are commonly made in a cylindrical shape, being formed in and subsequently protected by an iron casing, of which one form is shown in vertical section at Fig. 110. This represents a cast-iron cylinder composed of rings built up in sufficient number to reach the required height. Unless the bottom of the pier rests upon rock, a greater base area will be required than the area of the concrete column above, and to obtain this the lower rings of the cylinder are made of larger diameter than the upper. The bottom ring has a sharp lower edge $E\,E$ to facilitate its penetration of the soil. The rings D, resting upon the cutting ring, should be kept of the same diameter up to ground level, where a reduction of diameter is made by a conical ring C; of the same diameter as the top of this ring are the rings A and B above it. The top ring A is plain at

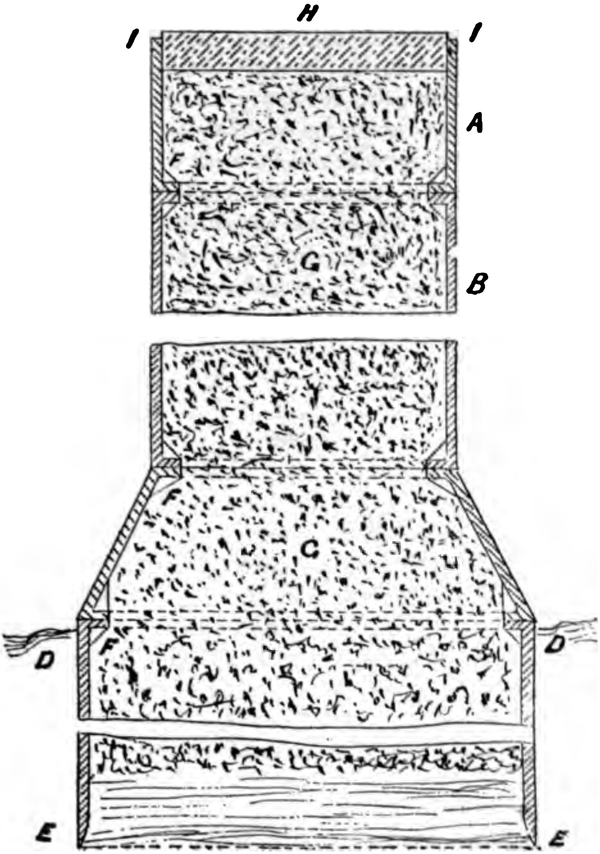

Fig. 110.

its upper edge, and the bearing-stone *H* rises 2 or 3 inches above it, so that no superincumbent load comes upon its upper edge. All the rings are made with internal flanges *F* bolted together as shown. External flanges upon the part to be sunk into the ground are inadmissible; the interior *G G* of the casing is filled up with concrete, brick, or masonry, or a combination of them, but must always be finished off at the top with masonry *H*.

Three methods of sinking these cyclinders have been used: the vacuum; compressed air and loading; and excavation and loading without compressed air. In the first a large vessel is exhausted of air and suddenly put in connection with the cylinder, which, becoming partially exhausted, is subjected to the superior air pressure above it tending to drive it down; of course the bottom must be in air-tight soil. This process is ingenious but unsatisfactory.

In the second method the cyclinder is charged with compressed air to keep water out, and allow men to excavate the soil beneath the cylinder; this process is eminently successful until great depths are reached, when the density of the air requisite to exclude the water interferes with the normal respiration of the workmen, who in such cases can only work continuously for short periods of time. The admission and exit of men and materials is managed through a system of air-locks on the top. The principle is simple, and involves only the use of an air-tight chamber which can be put into communication with the outer air and with the interior of the cylinder alternately. The necessary air-pressure in the cylinder is maintained by air-compressing pumps. If a man has to enter the cylinder the outer door of the air-lock is opened, and he enters that, closing the outer door air-tight. By opening a valve, air is then allowed to flow from the cylinder into the air-lock until the pressure

is the same in both, after which a door of communication lets the man through into the cylinder itself. In leaving the cylinder the course is reversed; the air being of equal pressure in cylinder and air-lock, the man passes from the former into the latter, and the door of communication is closed; the compressed air in the lock is then let out into the outer air, and when it has escaped the outer door is opened. The alteration in the pressure of air in the lock must be sufficiently slow, to prevent the dangerous effects that sudden changes of pressure would bring about, as the internal pressure in the human body has to be adjusted to the external. When materials only are passing through, this precaution is unnecessary.

The joints in the casing must be accurately faced, and rings of canvas smeared with white and red lead bolted between to keep them air-tight; the flanges, *F F*, have brackets between the bolts to strengthen them.

As soon as the cylinder, which is properly weighted at the top, has sunk to the required depth, the bottom ring can be made water-tight by a layer of concrete, and this having set the air-locks may be removed, and the rest of the work done under normal air pressure. The concrete used must be of good and solid quality, 6 parts gravel to 1 of hydraulic lime makes a good material. If the cylinders are sufficiently small to require bracing together, lugs are to be cast upon the upper rings for the connection of the bracing bars.

Another method of sinking cylinders in waterways is by excavating the soil within the weighted cylinder by means of the "scoop and bag." This implement has a sharp spoon-shaped cutting edge with a bag behind it, into which the soil dug by the edge falls; this is operated from above, and so can be worked under water, and no compressed air is required. *Hydraulic* concrete dropped through the water

in the cylinder will make a water-tight bottom, and when that has set the water can be pumped out and the filling in completed.

For perfection of method, the course taken in the construction of the centre pier of the Saltash Bridge, Cornwall, is worthy of careful study. The lower part of this pier consists of a cylinder of solid masonry 36 feet in diameter and about 100 feet high; above it are 4 huge cast-iron columns, upon which the main arch and chain-girders rest. It is with the lower part only of the pier I shall here deal.

A wrought-iron plate cylinder 37 feet in diameter and 100 feet high was made, closed at the top and floated over the site of the proposed pier; when it was judged from its angle of flotation to be in the right direction it was allowed to sink by letting the air out; it fell about 10 inches out of centre, but was again floated and dropped within 1 inch of the exact centre proposed for the pier.

The top of the cylinder being removed, a diving bell was let down inside it, and in this the men worked, there being trunks running up to the surface for entrance and exit and removal of the excavated materials.

Thus the cylinder was sunk through 30 feet of alluvial soil to the solid rock, into which it was built water-tight. The bell and its appurtenances was then cleared away, the cylinder pumped out, and the mason set to work to build up the masonry pier within. After the completion of the building work, the cylinder plates were cut asunder by divers, and brought away.

For piers constructed in iron encasements we are not confined to the cylindrical form. Small iron piles grooved on each side can be driven into the river bed, and flat or curved plates slid down the grooves, and having sharp edges to penetrate the soil, and thus an iron coffer-dam is

formed of any shape desired, for the grooves in the sides of the piles can be put in any position round the circumference.

A solid foundation on loose soil may sometimes be formed by driving piles in at small distances apart and leaving their heads projecting up about 5 or 6 feet above the ground level, and then making this height up solid with concrete. The idea is that the concrete will spread the load over the loose soil, and that the piles will prevent lateral movement of the concrete.

In some instances, a shifty sandy river-bed has been made into a good foundation by building a dam across the river up to bed level both above and below bridge, and concreting over to prevent the sand between from being washed out by scour or floods.

It has been proposed to convert sandy soil into a solid foundation by injecting lime or cement into it through tubes sunk down to the desired depth, and sufficiently near together for the lime to be disseminated continuously through the mass, the tubes being raised as each layer is thoroughly saturated, but I do not know whether the process has proved satisfactory in practice.

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